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CONSTRUCTIONAL ENGINEERING

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MAY, 1955. Vol. L, No. 5

FIFTIETH YEAR OF PUBLICATION

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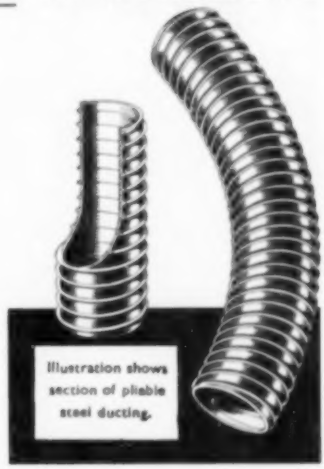

Illustration shows section of pliable steel ducting.

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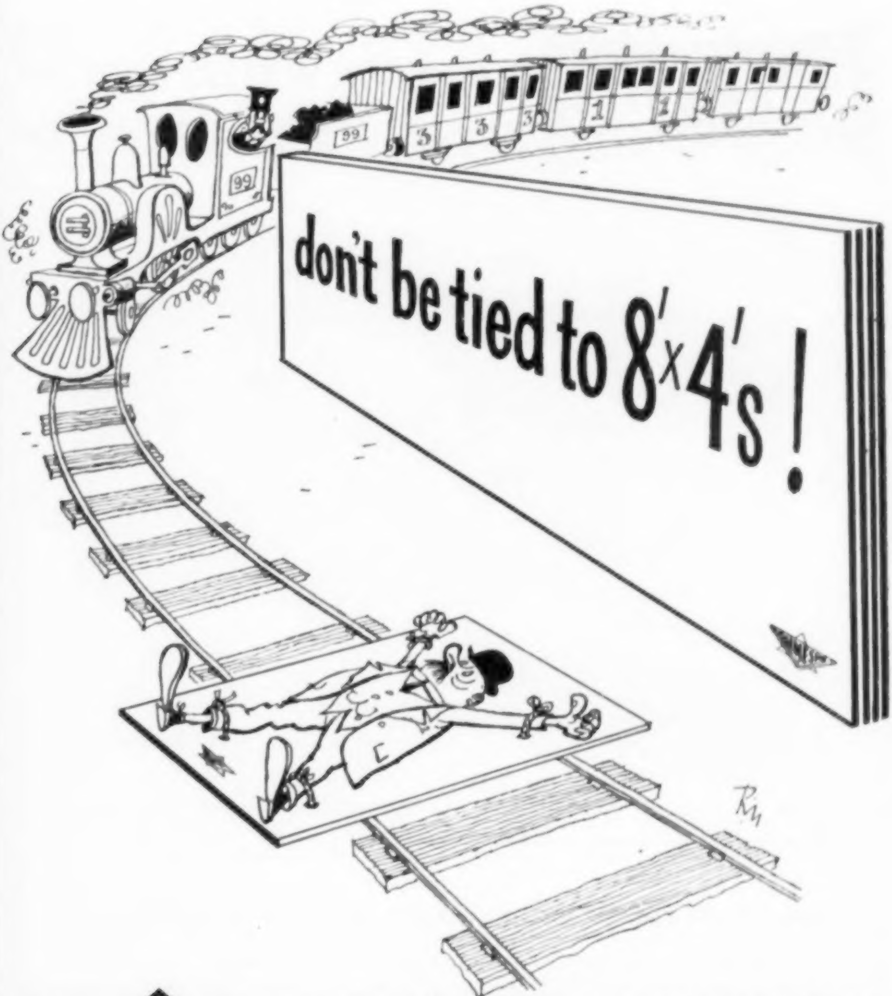
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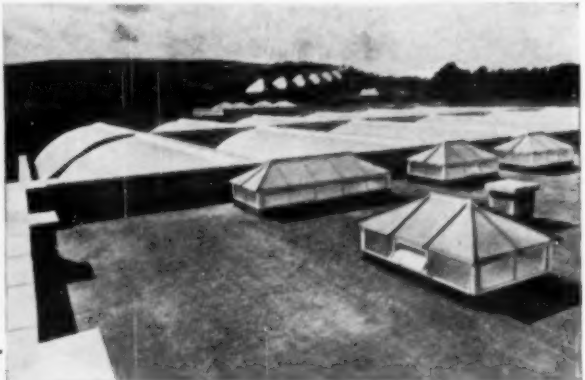

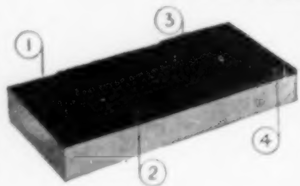

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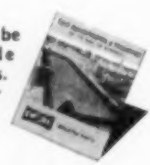

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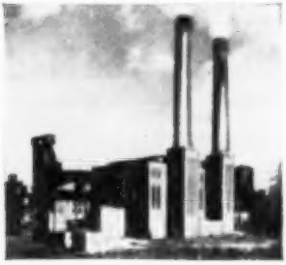

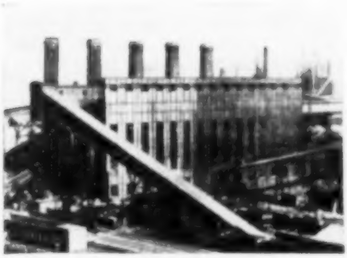

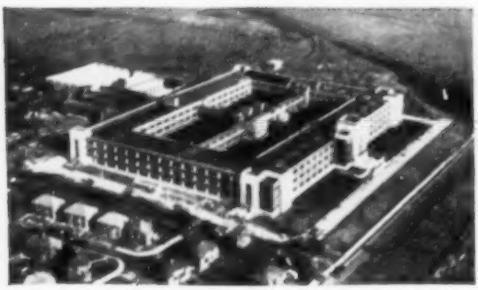

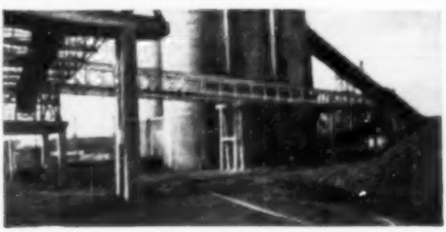

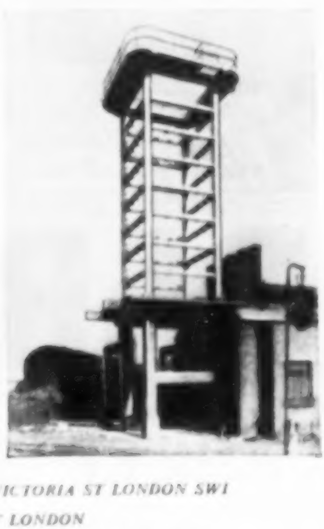

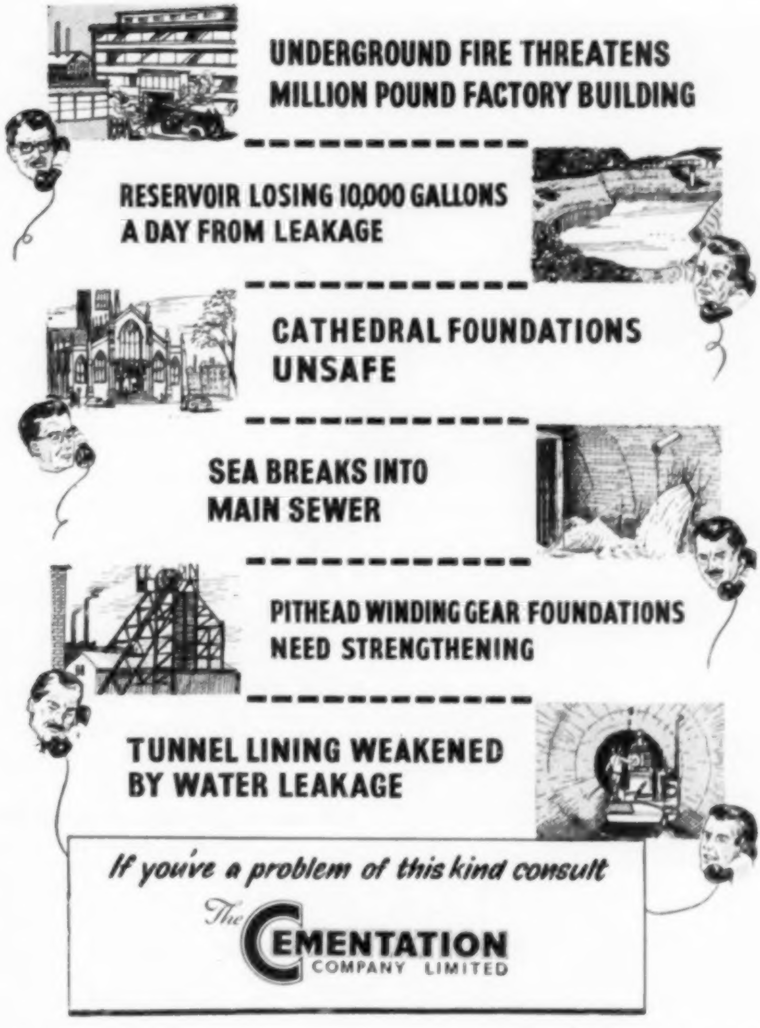
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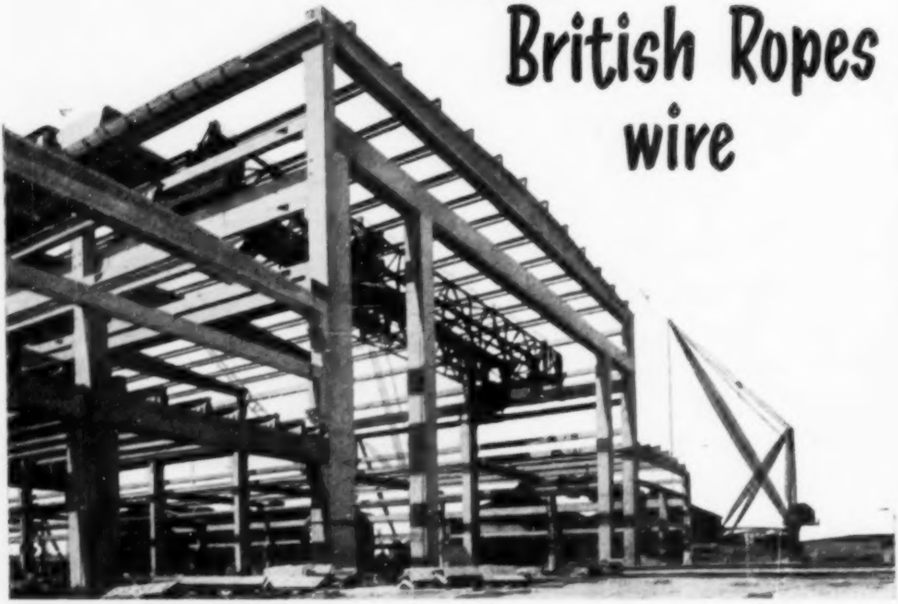

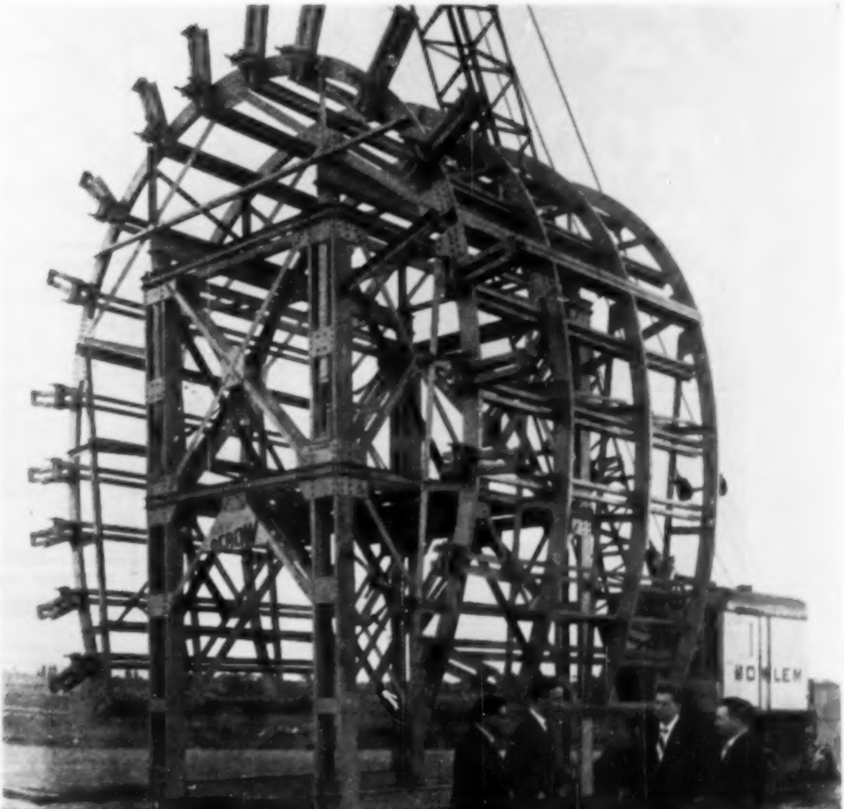
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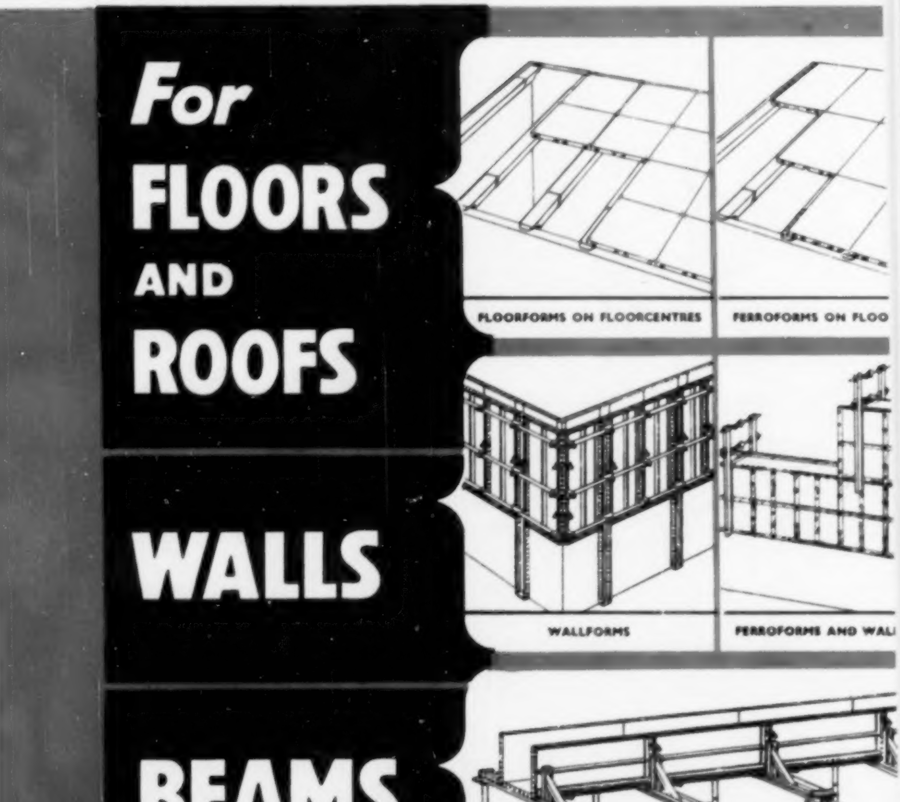
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Volume L, No. 5. LONDON, MAY, 1955.

EDITORIAL NOTES

A Proposal for Nationalising Construction.

FOR some years past the building trades unions have demanded the nationalisation of the industry. The reason is, of course, the benefits that nationalisation would bring to their members, but the claim has always been allied to an allegation that the industry in private ownership is entirely inefficient and that real efficiency can come only from ownership by the State. A proposal for the nationalisation of the industry has now been put forward in a report entitled "Policy for the Building Industry" issued by the Fabian Society. It is proposed that the Ministry of Works be responsible for all building works, that a Public Construction Corporation be formed to carry out all building and civil engineering work costing more than £5000, that construction units be formed to undertake the work, that a plant department be formed to hire plant to the construction units, that all materials and plant be purchased directly from the manufacturers, and that there be more research, more standardisation, and more codes of practice. Works costing less than £5000 would be left to private enterprise, and the activities of builders' merchants would be confined to supplying these small works. The builder, who merely assembles specified materials, is blamed for not being "research minded", presumably because he seldom presumes to tell the architect, the engineer, or the specialist subcontractor what materials he should specify or use. It is said, however, that "the traditional relationship between the building owners and their professional advisers is to be unchanged".

The reasons advanced in favour of nationalisation and of the expected beneficent results include all the arguments that have been proved to be entirely fallacious in the case of industries already acquired by the State. The report quotes the findings of committees to the effect that whereas the number of people employed in the industry is about the same as in the year 1938 the "efficiency" of the industry had declined by 25 per cent. in 1949. This decline in efficiency is said to be the result of delays in providing complete instructions before work starts, conditions of employment, scarcities of materials, the lack of building research, the need for more standardisation, "distrust" between architects and builders and between builders and trades unions, and the cost of submitting tenders. Most of these factors were, however, present in 1938 and all have improved since. There have been fewer disputes in the building trade than in any of the nationalised industries. More money has been spent on research.

Many more standards have been issued. More work is prefabricated. More plant is used. The cost of submitting tenders has been reduced by the increasing practice of asking selected firms only to tender. How, then, can these improvements have resulted in less efficiency? Of course they have not done so.

The only true criterion of the efficiency of an industry is the amount of work done per year per man employed and its cost, and the amount of work has indeed sadly declined. The causes, however, are the fewer working hours, the low standard insisted upon by the trades unions as a fair day's work, longer holidays, stops for tea-drinking, and idleness during working hours. These factors are, in present conditions, beyond the employers' control. One of the basic ideas underlying claims for nationalisation was the existence of an ideal man who would work harder and better if he knew he was working for the State than he would do if he were working for a private employer, and this conception still persists in spite of experience in the industries already nationalised. The truth seems to be that, in the minds of those employed in State-owned industries, nationalisation is synonymous with syndicalism: that when an industry is acquired by the State it belongs only to those employed in it, whose sole aim is to improve their own position at the expense of the rest of the nation. This ideal man does not exist, and certainly restrictive practices do not end when an industry is nationalised; on the contrary, experience proves that they increase.

Much importance is attached to greater efficiency, but nowhere is there any suggestion that there might be any competition. The proposal is that selected existing businesses would be purchased and handed over to a "construction unit" with a staff of about 500 to do the work required within a radius of ten miles of its depot, and neighbouring units would interchange their men as required to deal with the work in the areas of the different units. It seems that the unselected firms would be put out of business without compensation. All the men would be permanent employees. It is difficult to see how efficiency would be increased or costs reduced without the incentive of competition, and with the payment for the idleness that would be unavoidable if the men required at peak periods were also on the pay-roll in slack periods in an industry where the demand fluctuates so much as it does in building and civil engineering.

Architects and engineers are not at present included in the scheme. They would "prepare the plans jointly with a construction unit technician". However, if we may again accept experience as a guide, it would not be long before the Public Construction Corporation or its units set up their own architectural and engineering departments, as have the Ministries of Works, Health, Education, Housing and Local Government, the Services, and most local authorities, at costs that are quite unknown. It is noteworthy that the building owner, his professional adviser, and the "construction unit technician" are described as "a triumvirate who would jointly prepare all the necessary drawings and documents". Surely triumvirate is an unfortunate word to use. The first triumvirs were Pompey, Crassus, and Caesar. Caesar secured the support of Pompey and Crassus by bribery and threats, and thereby gained the Consulship. Within the next few years Pompey was murdered, Crassus committed suicide, and Caesar's dictatorship ended when he was murdered in the Senate. The proposed triumvirs in the building industry might well take note of the retribution that overtook the leader and collaborators in the first triumvirate.

Design of Helical Staircases.

By JACQUES S. COHEN.

This article gives a method of designing helical beams with a non-uniformly distributed load $w(s)$ and a non-uniformly distributed bending moment $m(s)$, both per unit length of the curve. The general differential equations of equilibrium of an element ds of the arc of any twisted curved beam related to three axes, loaded in this manner, are derived from Dr. Panayotounacos's "Analysis of any Twisted Curved Beams Loaded in any Direction"* and applied to the design of a staircase.

General Differential Equations of Equilibrium.

Consider any twisted curve related to the system of co-ordinates $Oxyz$ by

$$x = x(s) \; ; \; y = y(s) \; ; \; z = z(s) \qquad . \quad . \quad . \quad . \quad (1)$$

The element of the arc A_0A is ds and there are three principal lines at A_0, namely, a tangent T_0, a normal N_0, and a bi-normal B_0. At A there are similar lines T, N, and B (*Fig.* 1). Line A_0N_0 meets line AN in P, the centre of curvature; PA, the radius of curvature, is

$$\rho = \frac{ds}{d\omega} \qquad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

By analogy P′, where lines A_0B_0 and AB meet, is the centre of torsion; therefore P′A, the radius of torsion, is

$$\tau = \frac{ds}{d\eta} \qquad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

From the geometry of three dimensions † the direction-cosines of the three principal lines are—for T: α, β, and γ; for N: l, m, and n; for B: λ, μ, and ν. Values for these are given by equations (4) and are related by the Frenet-Serret formulæ (5).

From (2) and (3) and from *Fig.* 1 the cosines of the angles formed by the principal lines at points A_0 and A are as in (6).

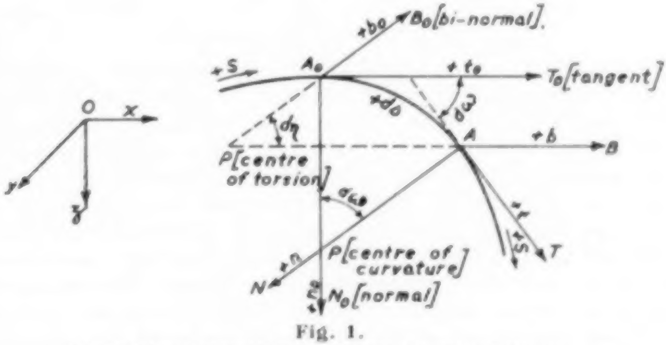

Fig. 1.

* Journal of the Technical Chamber of Greece, Nos. 295, 296, 310, 313.

† See "A Treatise on Analytic Geometry of Three Dimensions," by G. Salmon.

$$\left.\begin{aligned}
&\alpha = \frac{dx}{ds}; \quad \beta = \frac{dy}{ds}; \quad \gamma = \frac{dz}{ds}; \\
&l = \rho\frac{d^2x}{ds^2}; \quad m = \rho\frac{d^2y}{ds^2}; \quad n = \rho\frac{d^2z}{ds^2}. \\
&\lambda = \rho\left(\frac{dy}{ds}\cdot\frac{d^2z}{ds^2} - \frac{dz}{ds}\cdot\frac{d^2y}{ds^2}\right); \quad \mu = \rho\left(\frac{dz}{ds}\cdot\frac{d^2x}{ds^2} - \frac{dx}{ds}\cdot\frac{d^2z}{ds^2}\right); \\
&\nu = \rho\left(\frac{dx}{ds}\cdot\frac{d^2y}{ds^2} - \frac{dy}{ds}\cdot\frac{d^2x}{ds^2}\right)
\end{aligned}\right\} \quad \ldots \quad (4)$$

$$\begin{aligned}
&\frac{d\alpha}{ds} = \frac{l}{\rho}; \quad \frac{d\beta}{ds} = \frac{m}{\rho}; \quad \frac{d\gamma}{ds} = \frac{n}{\rho}; \quad \frac{d\lambda}{ds} = -\frac{l}{\tau}; \quad \frac{d\mu}{ds} = -\frac{m}{\tau}; \quad \frac{d\nu}{ds} = -\frac{n}{\tau}; \\
&\frac{dl}{ds} = -\frac{\alpha}{\rho} + \frac{\lambda}{\tau}; \quad \frac{dm}{ds} = -\frac{\beta}{\rho} + \frac{\mu}{\tau}; \quad \frac{dn}{ds} = -\frac{\gamma}{\rho} + \frac{\nu}{\tau} \quad \ldots \quad (5)
\end{aligned}$$

$$\left.\begin{aligned}
&t_t = \cos(\bar{t}_0, \bar{t}) = 1; \quad t_n = \cos(\bar{t}_0, \bar{n}) = -\frac{ds}{\rho}; \quad t_b = \cos(\bar{t}_0, \bar{b}) = 0; \\
&n_t = \cos(\bar{n}_0, \bar{t}) = -t_n = \frac{ds}{\rho}; \quad n_n = \cos(\bar{n}_0, \bar{n}) = 1; \quad n_b = \cos(\bar{n}_0, \bar{b}) = \frac{-ds}{\tau}; \\
&b_t = \cos(\bar{b}_0, \bar{t}) = t_b = 0; \quad b_n = \cos(\bar{b}_0, \bar{n}) = -n_b = \frac{ds}{\tau}; \quad b_b = \cos(\bar{b}_0, \bar{b}) = 1
\end{aligned}\right\} \quad (6)$$

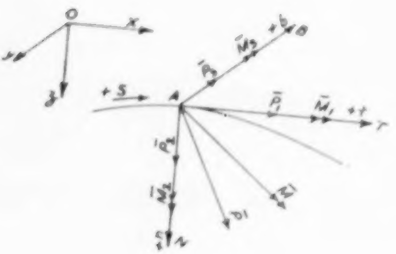

Fig. 2.

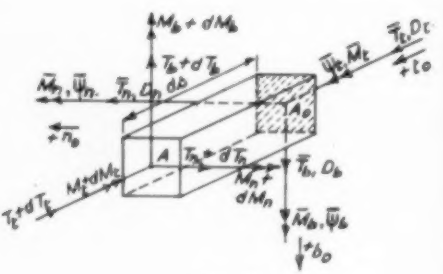

Fig. 3.

Consider point A (*Fig.* 2) on the axis of a twisted curved beam with equally or unequally distributed load over the beam. At A this load is represented by a resultant force $\bar{P}$ and a resultant moment $\bar{M}$ which can be replaced by their projections on the three principal lines at A; the components are forces $\bar{P}_1$, $\bar{P}_2$, $\bar{P}_3$, and moments $\bar{M}_1$, $\bar{M}_2$, and $\bar{M}_3$. The internal forces and moments caused are an axial force T_t (tensile or compressive), shearing forces T_n and T_b, a twisting moment M_t, and bending moments M_n and M_b. Also caused are displacements of the centroid of the section D_t, D_n, and D_b, and angular rotations ψ_t, ψ_n, ψ_b (*Fig.* 3). In the case where the principal lines coincide with the principal axes of inertia of the section, these are the main internal forces and moments of the section. Where this is not so the adjustments shown in *Fig.* 4 must be made to relate them to the principal axes of inertia of the section.

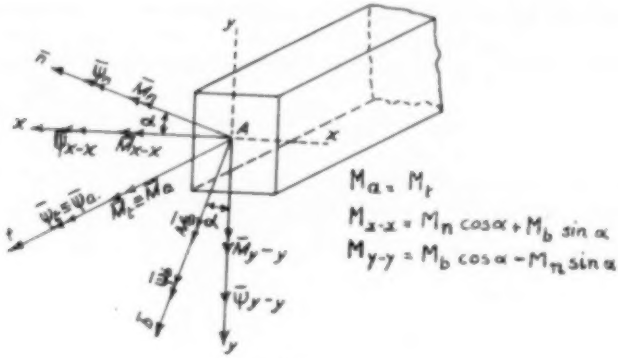

Fig. 4.

Sign conventions are shown in *Figs.* 1 to 4, the directions of the moments being indicated by double arrows. The element of a beam has a non-uniformly distributed load $w(s)$ per unit length of s, where $\bar{w} = w_t.\bar{t} + w_n\bar{n} + w_b\bar{b}$, and a non-uniformly distributed moment $m(s)$ per unit length of (s) where $\bar{m} = m_t\bar{t} + m_n\bar{n} + w_b\bar{b}$. The components of $w(s)$ and $m(s)$ about the axes t, $\bar{n}$, and b are also functions of the arc s, that is,

$$w_t = w_t(s)\,;\quad w_n = w_n(s)\,;\quad w_b = w_b(s)\,;\quad m_t = m_t(s)\,;\quad m_n = m_n(s)\,;\quad m_b = m_b(s)\,. \qquad (7)$$

The six conditions of equilibrium for the element about the axes found by substituting values from equations (6) are given in equations (8).

$$\left.\begin{aligned}
&T_t - (T_t + dT_t) + (T_n + dT_n)\frac{ds}{\rho} + w_t\,ds - w_n\,ds\frac{ds}{2\rho} = 0.\\
&T_n - (T_n + dT_n) - (T_t + dT_t)\frac{ds}{\rho} + (T_b + dT_b)\frac{ds}{\tau} + w_t\,ds\,\frac{ds}{2\rho} + w_n\,ds - w_b\,ds\frac{ds}{2\tau} = 0.\\
&T_b - (T_b + dT_b) - (T_n + dT_n)\frac{ds}{\tau} + w_b\,ds + w_n\,ds\frac{ds}{2\tau} = 0.\\
&M_t - (M_t + dM_t) + (M_n + dM_n)\frac{ds}{\rho} + w_b\,ds\,\rho\left(1 - \cos\frac{d\omega}{2}\right) -\\
&\qquad - (T_b + dT_b)\cos d\eta\,\rho(1 - \cos d\omega) + m_t\,ds - m_n\,ds\frac{ds}{2\rho} = 0.\\
&M_n - (M_n + dM_n) - (M_t + dM_t)\frac{ds}{\rho} + (M_b + dM_b)\frac{ds}{\tau} - w_b\,ds\,\rho\sin\frac{d\omega}{2} +\\
&\qquad + (T_b + dT_b)\cos d\eta\,\rho\sin d\omega + m_n\,ds + m_t\,ds\,\frac{ds}{2\rho} - m_b\,ds\,\frac{ds}{2\tau} = 0.\\
&M_b - (M_b + dM_b) - (M_n + dM_n)\frac{ds}{\tau} + \left(w_t - w_b\sin\frac{d\eta}{2}\right)\rho\left(1 - \cos\frac{d\omega}{2}\right)ds +\\
&\qquad + w_n\,ds\,\rho\sin\frac{d\omega}{2} - (T_n + dT_n)\rho\sin d\omega -\\
&\qquad - [T_t + dT_t - (T_b + dT_b)\sin d\eta]\rho(1 - \cos d\omega) + m_b\,ds + m_n\,ds\,\frac{ds}{2\tau} = 0.
\end{aligned}\right\} \quad (8)$$

Since $d\omega$ and $d\eta$ are infinitesimally small angles,

$$\cos d\omega = \cos d\eta = \cos\frac{d\omega}{2} = \cos\frac{d\eta}{2} = 1\,;\quad \sin d\omega = d\omega\,;\quad \sin\frac{d\omega}{2} = \frac{d\omega}{2}\,;$$

$$\sin d\eta = d\eta\,;\quad \text{and } \sin\frac{d\eta}{2} = \frac{d\eta}{2}.$$

Ignoring the infinitesimal angles of the second order, and taking into consideration the relationship of equations (2) and (3), equations (8) become

$$\frac{dT_t}{ds} = \frac{T_n}{\rho} + w_t\,;\quad \frac{dT_n}{ds} = -\frac{T_t}{\rho} + \frac{T_b}{\tau} + w_n\,;\quad \frac{dT_b}{ds} = -\frac{T_n}{\tau} + w_b\,;$$

$$\frac{dM_t}{ds} = \frac{M_n}{\rho} + m_t\,;\quad \frac{dM_n}{ds} = -\frac{M_t}{\rho} + \frac{M_b}{\tau} + T_b + m_n\,;\quad \frac{dM_b}{ds} = -\frac{M_n}{\tau} - T_n + m_b \quad . \quad (9)$$

The first three equations of (9), after differentiation and substitution, result in a linear differential equation of the third order with a second member of T_t.

$$\tau\rho\frac{d^3T_t}{ds^3} + \left[\frac{d(\tau.\rho)}{ds} + \tau\frac{d\rho}{ds}\right]\frac{d^2T_t}{ds^2} + \left(\frac{d\left[\tau\frac{d\rho}{ds}\right]}{ds} + \frac{\tau}{\rho} + \frac{\rho}{\tau}\right)\frac{dT_t}{ds} + \frac{d\left[\frac{\tau}{\rho}\right]}{ds}T_t$$
$$= \frac{d\left[\tau\frac{d(\rho w_t)}{ds}\right]}{ds} + \frac{\rho}{\tau}w_t + w_b + \frac{d(\tau w_n)}{ds} \quad . \quad (10)$$

The last three equations of (9), after differentiation and substitution, result also in a linear differential equation of the third order with a second member of M_t, as in (11).

$$\tau\rho\frac{d^3M_t}{ds^3} + \left[\frac{d(\tau.\rho)}{ds} + \tau\frac{d\rho}{ds}\right]\frac{d^2M_t}{ds^2} + \left(\frac{d\left(\tau\frac{d\rho}{ds}\right)}{ds} + \frac{\tau}{\rho} + \frac{\rho}{\tau}\right)\frac{dM_t}{ds} + \frac{d\left[\frac{\tau}{\rho}\right]}{ds}M_t$$
$$= \frac{d\left[\tau\frac{d(\rho m_t)}{ds}\right]}{ds} + \frac{\rho}{\tau}m_t + (m_b - T_n) + \frac{d[\tau(T_b + m_n)]}{ds}. \quad . \quad (11)$$

When values of T_t and M_t have been obtained, values of T_n, T_b, M_n, and M_b are derived from (9) and are given in (12a) to (12d).

$$T_n = \rho\frac{dT_t}{ds} - \rho w_t \quad . \quad . \quad . \quad . \quad . \quad . \quad (12a)$$

$$T_b = \tau\frac{d\left(\rho\frac{dT_t}{ds}\right)}{ds} - \tau\frac{d(\rho w_t)}{ds} + \frac{\tau}{\rho}T_t - \tau w_n \quad . \quad . \quad . \quad (12b)$$

$$M_n = \rho\frac{dM_t}{ds} - \rho m_t \quad . \quad . \quad . \quad . \quad . \quad . \quad (12c)$$

$$M_b = \tau\frac{d\left(\rho\frac{dM_t}{ds}\right)}{ds} - \tau\frac{d(\rho m_t)}{ds} + \frac{\tau}{\rho}M_t - \tau(m_n + T_b) \quad . \quad . \quad (12d)$$

Equations (10), (11), and (12) give the values of T_t, T_n, T_b, M_t, M_n, and M_b at any section of a twisted curved beam subjected to a non-uniformly distributed load and a non-uniformly distributed bending moment.

Application of the Equations.

The equations for a helix about the axes xyzO (*Fig.* 5) are

$$x = a\cos\theta\,;\quad y = a\sin\theta\,;\quad z = c.\theta\,. \qquad (13)$$

where θ, the polar angle, is the independent variable of the equation, a is the radius of the cylinder, and c is a constant. From the development of the cylinder (*Fig.* 5), $c = \dfrac{h}{2\pi}$, $\phi = \cot^{-1}\dfrac{h}{2\pi a} = \cot^{-1}\dfrac{c}{a}$ and is a constant, and $\cot\phi = \dfrac{c}{a}$. Differentiating, $dx = -a\sin\theta d\theta$, $dy = a\cos\theta d\theta$, and $dz = cd\theta$. From

$$ds = \sqrt{dx^2 + dy^2 + dz^2},$$

the length of the arc of the helix (s) from A_0 to A is $s = \sqrt{c^2 + a^2}.\theta$.

$$s = a\sqrt{\frac{c^2}{a^2} + 1}.\theta = a\sqrt{\cot^2\phi + 1}.\theta = \frac{a}{\sin\phi}\theta = \frac{\theta}{K} \qquad (14)$$

where $K = \dfrac{\sin\phi}{a}$ and is a constant. With s as an independent variable, (13) becomes

$$x = a\cos(Ks),\ y = a\sin(Ks),\ z = cKs \qquad (15)$$

Equations (4) and (5) when applied to a helix are as in (16).

$$\left.\begin{array}{l} \alpha = -\sin\phi\sin(Ks)\,;\ \beta = \sin\phi\cos(Ks)\,;\ \gamma = \cos\phi \\ l = -\cos(Ks)\,;\ m = -\sin(Ks)\,;\ n = 0 \\ \lambda = \cos\phi\sin(Ks)\,;\ \mu = -\cos\phi\cos(Ks)\,;\ \nu = \sin\phi. \\ \rho = \dfrac{a}{\sin^2\phi}\text{ and is a constant}\,;\ \tau = \dfrac{2a}{\sin 2\phi}\text{ and is a constant.} \end{array}\right\} \qquad (16)$$

The cosines of the angles between the principal lines at two points close together are obtained from (6) and are given in (17).

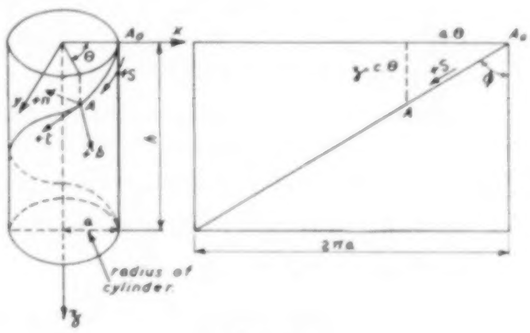

Fig. 5.

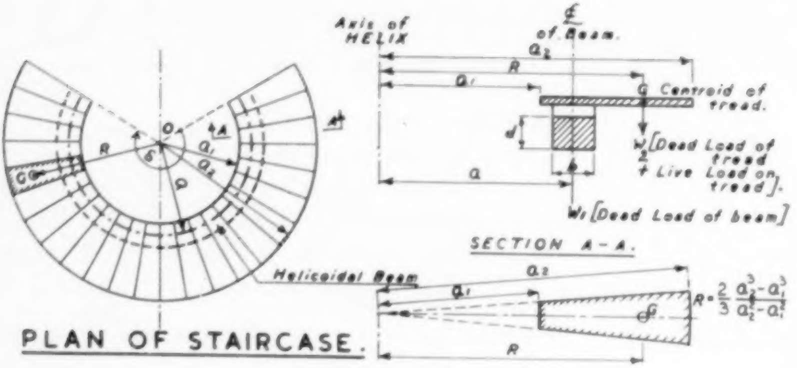

Fig. 6.

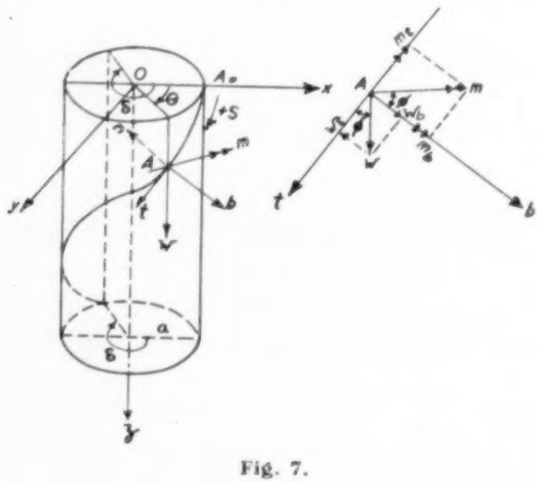

Fig. 7.

$$\left.\begin{array}{l} t_t = \cos(t_0, t) = 1,\ t_n = \cos(t_0, \bar{n}) = -\sin\phi\, K\, ds,\ t_b = \cos(t_0, b) = 0 \\ n_t = \cos(\bar{n}_0, t) = \sin\phi\, K\, ds,\ n_n = \cos(\bar{n}_0, \bar{n}) = 1,\ n_b = \cos(\bar{n}_0, b) = -\cos\phi\, K\, ds \\ b_t = \cos(b_0, t) = 0,\ b_n = \cos(b_0, \bar{n}) = \cos\phi\, K\, ds,\ b_b = \cos(b_0, b) = 1. \end{array}\right\} \quad (17)$$

Consider the staircase shown in *Figs.* 6 and 7. At any point A on the centre-line of the beam there is a vertical uniformly-distributed load of w lb. per foot length of the helix; $w = \frac{W_1 + W_2}{s} = \frac{W}{s}$, and a uniformly-distributed moment of m ft.-lb. per foot length of the helix due to the load applied at the centroid of each tread: $m = \frac{W_2}{s}(R - a)$.

By relating the load and bending moment to the three principal lines,

$$w_t = w\cos\phi\,;\ w_n = 0\,;\ w_b = w\sin\phi\,;\ m_t = -m\sin\phi\,;\ m_n = 0\,;\ m_b = m\cos\phi\ . \quad (18)$$

where w_t, w_b, m_t, and m_b are constants. Therefore

$$\frac{dw_t}{ds} = \frac{dw_b}{ds} = \frac{d^2w_t}{ds^2} = \frac{d^2w_b}{ds^2} = 0 \text{ and } \frac{dm_t}{ds} = \frac{dm_b}{ds} = \frac{d^2m_t}{ds^2} = \frac{d^2m_b}{ds^2} = 0 \quad . \quad (19)$$

Applying the general equations (10) and (11) and rearranging (16), (18), and (19),

$$\frac{a^2}{\sin^2\phi}\cdot\frac{d^3T_t}{ds^3} + \frac{dT_t}{ds} = w\cos\phi \quad . \quad . \quad . \quad . \quad (20)$$

$$\frac{a^2}{\sin^2\phi}\cdot\frac{d^3M_t}{ds^3} + \frac{dM_t}{ds} = -\sin\phi\cos\phi T_n + a\frac{dT_b}{ds} \quad . \quad . \quad (21)$$

Equations (20) and (21) are linear differential equations of the third order with constant coefficients and a second member. The roots of the characteristic equations of (20) and (21) are $r_1 = 0$, $r_2 = i\frac{\sin\phi}{a}$, and $r_3 = -i\frac{\sin\phi}{a}$. Therefore, if T_{t_1} and M_{t_1} are particular solutions of (20) and (21), the general solutions are

$$T_t = C_1 + C_2\sin(Ks) + C_3\cos(Ks) + T_{t_1} \quad . \quad . \quad . \quad (20a)$$

$$M_t = C_4 + C_5\sin(Ks) + C_6\cos(Ks) + M_{t_1} \quad . \quad . \quad (21a)$$

The particular solution of (20) is $T_1 = w\cos\phi . s$. Therefore equations (20a), (12a), and (12b) become

$$\left.\begin{aligned} T_t &= C_1 + C_2\sin(Ks) + C_3\cos(Ks) + w\cos\phi . s \\ T_n &= \frac{1}{\sin\phi}[C_2\cos(Ks) - C_3\sin(Ks)] \\ T_b &= -\cos\phi[C_2\sin(Ks) + C_3\cos(Ks) + \tan\phi(C_1 + w\cos\phi . s)] \end{aligned}\right\} \quad . \quad (22)$$

Substituting these values of T_n and T_b in (21),

$$\frac{a^2}{\sin^2\phi}\,\frac{d^3M_t}{ds^3} + \frac{dM_t}{ds} = -2\cos\phi[C_2\cos(Ks) - C_3\sin(Ks)] + aw\sin\phi.$$

The particular solution of (21a) is

$$M_{t_1} = \cos\phi[C_2\cos(Ks) - C_3\sin(Ks)]s + saw\sin\phi.$$

Therefore equations (21a), (12c), and (12d), with values from (14), (16), (18), (19), and (22) substituted in them, give (23).

$$\left.\begin{aligned} M_t &= C_4 + C_5\sin(Ks) + C_6\cos(Ks) + \\ &\qquad + s\cos\phi[C_2\cos(Ks) - C_3\sin(Ks)] + saw\sin\phi. \\ M_n &= \frac{1}{\sin\phi}[C_5\cos(Ks) - C_6\sin(Ks)] + \frac{a\cos\phi}{\sin^2\phi}[C_2\{\cos(Ks) - Ks\sin(Ks)\} - \\ &\qquad - C_3\{\sin(Ks) + Ks\cos(Ks)\}] + \frac{wa^2}{\sin\phi} + \frac{ma}{\sin\phi} \\ M_b &= -\cot\phi[C_5\sin(Ks) + C_6\cos(Ks)] + \\ &\qquad + \tan\phi C_4 + s\frac{\cos^2\phi}{\sin\phi}[-C_2\cos(Ks) + C_3\sin(Ks)] - \\ &\qquad - \frac{a}{\sin^2\phi}[C_2\sin(Ks) + C_3\cos(Ks)] - \frac{a}{\cos^2\phi}C_1 - saw\cos\phi. \end{aligned}\right\} \quad (23)$$

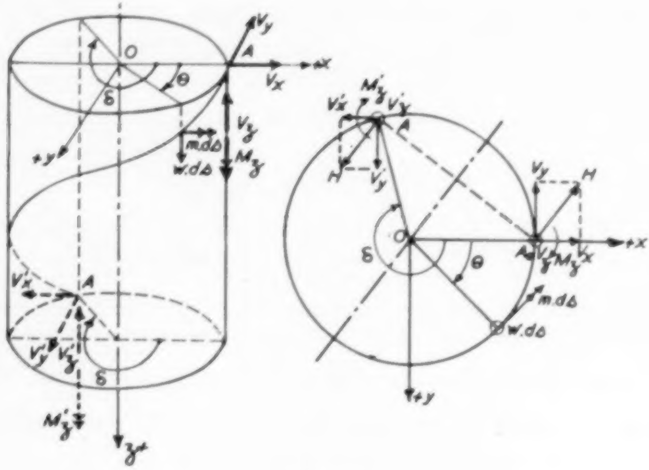

Fig. 8.

Thus equations (22) and (23) give the values of T_t, T_n, T_b, M_t, M_n, and M_b as functions of s. Changing the variable from s to the polar angle θ,

$$s = \frac{a}{\sin\phi}\theta,\ ds = \frac{a}{\sin\phi}d\theta,\ ds^2 = \frac{a^2}{\sin^2\phi}d\theta^2,\ ds^3 = \frac{a^3}{\sin^3\phi}d\theta^3.$$

Substituting in equations (22) and (23), we get (24).

$$\left.\begin{aligned}
T_t &= C_1 + C_2\sin\theta + C_3\cos\theta + aw\cot\phi.\theta \\
T_n &= \frac{1}{\sin\phi}[C_2\cos\theta - C_3\sin\theta] \\
T_b &= -\cot\phi(C_2\sin\theta + C_3\cos\theta) + \tan\phi C_1 + aw\theta \\
M_t &= C_4 + C_5\sin\theta + C_6\cos\theta + a\theta\cot\phi(C_2\cos\theta - C_3\sin\theta) + wa^2\theta \\
M_n &= \frac{1}{\sin\phi}[C_5\cos\theta - C_6\sin\theta + a\cot\phi\{C_2(\cos\theta - \theta\sin\theta) \\
&\qquad - C_3(\sin\theta + \theta\cos\theta)\} + wa^2 + ma] \\
M_b &= -\cot\phi(C_5\sin\theta + C_6\cos\theta) + \tan\phi C_4 + a\cot^2\phi.\theta(-C_2\cos\theta + C_3\sin\theta) \\
&\qquad - \frac{a}{\sin^2\phi}(C_2\sin\theta + C_3\cos\theta) - \frac{a}{\cos^2\phi}C_1 - wa^2\cot\phi.\theta
\end{aligned}\right\} \quad (24)$$

If the values of the internal forces and moments for one section are known, for example at the support, the values of the constants C_1, C_2, C_3, etc., may be calculated from (24).

The solution depends on the restraint at the ends of the beam. For determinate beams (that is cantilevers, simply-supported beams, and in some cases of helical beams with three supports), the forces and moments at the supports may be found from the equations of equilibrium. For indeterminate beams the equations of equilibrium are not sufficient, and equations of deformation and

angular rotation at any point are also required. In the present article, only beams simply supported at their ends are considered. In such beams (*Fig.* 8) there are no moments about the horizontal axes Ox and Oy; therefore the projections of the forces and moments acting at the supports are

At A_0: V_x, V_y, V_z, and M_z. At A: V_x', V_y', V_z' and M_z'.

The division of the vertical load between A_0 and A depends on the relative stiffness of the upper and lower supports; it is assumed that $V_z = V_z'$ and $M_z = M_z'$.

Resolving the forces and calculating the moments about the three axes passing through A, we get (25).

$$\left.\begin{aligned}
&V_x - V_x' = 0, \quad -V_y + V'_y = 0, \quad \int_0^\delta w\,ds - 2V_z = 0,\\
&\int_0^\delta m\,ds\sin\theta - V_z(y_0 - y_A) + \int_0^\delta w\,ds(y - y_A) + V_y(z_0 - z_A) = 0\\
-&\int_0^\delta m\,ds\cos\theta + V_z(x_0 - x_A) - \int_0^\delta w\,ds(x - x_A) + V_x(z_0 - z_A) = 0\\
&2M_z - V_x(y_0 - y_A) - V_y(x_0 - x_A) = 0
\end{aligned}\right\} \quad (25)$$

From equations (13) and (14) we get (26).

$$\left.\begin{aligned}
&x_0 - x_A = a\cos 0 - a\cos\delta = a(1 - \cos\delta),\\
&\qquad y_0 - y_A = a\sin 0 - a\sin\delta = -a\sin\delta,\\
&z_0 - z_A = 0 - a\cot\phi.\delta = -a\cot\phi.\delta, \; x - x_A = a(\cos\theta - \cos\delta),\\
&\qquad y - y_A = a(\sin\theta - \sin\delta),\\
&\int_0^\delta w\,ds = \int_0^\delta \frac{wa}{\sin\phi}d\theta = \frac{wa}{\sin\phi}\delta,\\
&\int_0^\delta w\,ds(y - y_A) = \int_0^\delta \frac{wa^2}{\sin\phi}(\sin\theta - \sin\delta)d\theta = \frac{wa^2}{\sin\phi}[(1 - \cos\delta) - \delta\sin\delta]\\
&\int_0^\delta w\,ds(x - x_A) = \int_0^\delta \frac{wa^2}{\sin\phi}[\cos\theta - \cos\delta]d\theta = \frac{wa^2}{\sin\phi}(\sin\delta - \delta\cos\delta)\\
&\int_0^\delta m\,ds\sin\theta = \int_0^\delta \frac{ma}{\sin\phi}\sin\theta\,d\theta = \frac{ma}{\sin\phi}(1 - \cos\delta)\\
&\int_0^\delta m\,ds\cos\theta = \int_0^\delta \frac{ma}{\sin\phi}\cos\theta\,d\theta = \frac{ma}{\sin\phi}\sin\delta.
\end{aligned}\right\} \quad (26)$$

Substituting the values of (26) in (25), we get (27).

$$\left.\begin{aligned}
V_x = V_x' &= \frac{-m}{\cos\phi}\frac{\sin\delta}{\delta} + \frac{wa}{\cos\phi}\left(\frac{1 + \cos\delta}{2} - \frac{\sin\delta}{\delta}\right)\\
V_y = V_y' &= \frac{m}{\cos\phi}\frac{1 - \cos\delta}{\delta} + \frac{wa}{\cos\phi}\left(\frac{1 - \cos\delta}{\delta} - \frac{\sin\delta}{2}\right)\\
V_z = V_z' &= \frac{wa}{2\sin\phi}\delta\\
M_z = M_z' &= \frac{ma}{\cos\phi}\frac{1 - \cos\delta}{\delta} + \frac{wa^2}{\cos\phi}\left(\frac{1 - \cos\delta}{\delta} - \frac{\sin\delta}{2}\right)
\end{aligned}\right\} \quad (27)$$

The values of V_y, V_y', V_z, V_z', M_z, and M_z' are positive for any value of δ; therefore their directions are as shown in *Fig.* 8. V_x and V_x' for values of δ between 0 and π are negative and their directions are contrary to that shown, but for values of δ between π and 2π they are positive and in the directions shown.

The projections of V_x, V_y, V_z, and M_z on the three principal axes are obtained from (28).

$$\left.\begin{aligned} T_t &= \alpha V_x + \beta V_y + \gamma V_z\,; \quad T_n = lV_x + mV_y + nV_z\,; \\ &\qquad\qquad\qquad\qquad T_b = \lambda V_x + \mu V_y + \nu V_z \\ M_t &= \alpha M_x + \beta M_y + \gamma M_z\,; \quad M_n = lM_x + mM_y + nM_z\,; \\ &\qquad\qquad\qquad\qquad M_b = \lambda M_x + \mu M_y + \nu M_z \end{aligned}\right\} \quad . \quad . \quad (28)$$

At the origin of the helix A_0 from equation (16),

$$\begin{array}{lll} \alpha = 0 & l = -1 & \lambda = -0 \\ \beta = \sin\phi & m = 0 & \mu = \cos\phi \\ \gamma = \cos\phi & n = 0 & \nu = \sin\phi. \end{array}$$

Substituting these values in (24) and (28), when $\theta = 0$, we get (29).

$$\left.\begin{aligned} T_{t_0} &= \sin\phi V_y + \cos\phi V_z = C_1 + C_3 \\ T_{n_0} &= -V_x = \frac{C_2}{\sin\phi} \\ T_{b_0} &= -\cos\phi V_y + \sin\phi V_z = -\cot\phi C_3 + \tan\phi\, C_1 \\ M_{t_0} &= \cos\phi M_z = C_4 + C_6 \\ M_{n_0} &= 0 = C_5 + a\cot\phi C_2 + wa^2 + ma \\ M_{b_0} &= \sin\phi M_z = -\cot\phi C_6 + \tan\phi C_4 - \frac{a}{\sin^2\phi}C_3 - \frac{a}{\cos^2\phi}C_1 \end{aligned}\right\} \quad . \quad (29)$$

By substituting in (29) the values of V_x, V_y, V_z, M_z, the values of the constants are as in (30).

$$\left.\begin{aligned} C_1 &= -\frac{wa}{2}\delta\cot\phi\,; \quad C_2 = m\tan\phi\frac{\sin\delta}{\delta} - wa\tan\phi\left(\frac{1+\cos\delta}{2} - \frac{\sin\delta}{\delta}\right); \\ C_3 &= -m\tan\phi\frac{1-\cos\delta}{\delta} - wa\tan\phi\left[\frac{1-\cos\delta}{\delta} - \frac{\sin\delta}{2}\right] = -C_2\tan\frac{\delta}{2}\,; \\ C_4 &= -\frac{wa^2}{2}\delta\,; \quad C_5 = -wa^2 - ma - a\cot\phi C_2\,; \\ C_6 &= ma\frac{1-\cos\delta}{\delta} + wa^2\left(\frac{1-\cos\delta}{\delta} - \frac{\sin\delta}{2}\right) + \frac{wa^2}{2}\delta = -a\cot\phi C_3 - C_4 \end{aligned}\right\} \quad (30)$$

From (24) and (30) the shearing forces and bending moments about the principal axes, and the normal force and twisting moments, may be found for any point in a simply-supported helical beam. The values of T_{t_A}, T_{n_A}, T_{b_A}, M_{t_A}, M_{n_A}, and M_{b_A} at the other end of the beam can be obtained by resolving the forces and calculating the moments about the three axes passing through A_0; they are:

$$T_{t_A} = -T_{t0}\,; \quad T_{n_A} = T_{n0}\,; \quad T_{b_A} = -T_{b0}\,;$$

$$M_{t_A} = -M_{t0}\,; \quad M_{n_A} = M_{n0}\,; \text{ and } M_{b_A} = -M_{b0} \quad . \quad . \quad . \quad (31)$$

To make sure that there are no discrepancies in (24), the values obtained from

these equations must be the same as those obtained from (31). This check is made in the following numerical example.

Numerical Application.

Consider the staircase shown in *Fig.* 6 where $a = 2$ ft. 11 in., $a_1 = 1$ ft. 9 in., $a_2 = 5$ ft. 3 in., $\delta = \dfrac{4\pi}{3} = 240$ deg., and the height is 11 ft. 3 in. Timber treads 3 ft. 6 in. long and 2 in. thick are fixed to a reinforced concrete helical beam. The beam is 1 ft. $1\frac{1}{2}$ in. wide and $8\frac{1}{2}$ in. deep.

From equation (13), $c = \dfrac{h}{\delta} = \dfrac{3 \times 11{\cdot}25}{4\pi} = 2{\cdot}687$;

$$\cot\phi = \frac{c}{a} = \frac{2{\cdot}687}{2{\cdot}918} = 0{\cdot}922\,;\quad \phi = 47 \text{ deg. } 20 \text{ min.}\,;$$

$$\sin\phi = 0{\cdot}735\,;\quad \cos\phi = 0{\cdot}678\,;\quad \tan\phi = 1{\cdot}085.$$

From equation (14) $K = \dfrac{\sin\phi}{a} = \dfrac{0{\cdot}735}{2{\cdot}918} = 0{\cdot}252$;

$$s = \text{length of helix} = \frac{\delta}{K} = \frac{4\pi}{3 \times 0{\cdot}252} = 16{\cdot}63 \text{ ft.}\,;$$

$$R = \text{centroid of each thread} = \frac{2}{3} \times \frac{5{\cdot}25^3 - 1{\cdot}75^3}{5{\cdot}25^2 - 1{\cdot}75^2} = 3{\cdot}785 \text{ ft.}$$

The weight of the beam is 1915 lb. and the weight of treads and live load (60 lb. per square foot) is 3935 lb. (total, 5850 lb.). $w = \dfrac{5850}{16{\cdot}63} = 352$ lb. per foot length of the beam; $m = \dfrac{3935}{16{\cdot}63}(3{\cdot}785 - 2{\cdot}918) = 206$ ft.-lb. per foot length of the beam.

From equation (30) the constant values are

$C_1 = -5{\cdot}634w = -1983\,;\quad C_2 = -1{\cdot}445w - 0{\cdot}224m = -555\,;$
$C_3 = -2{\cdot}503w - 0{\cdot}388m = -961\,;\quad C_4 = -17{\cdot}83w = -6276\,;$
$C_5 = -4{\cdot}628w - 2{\cdot}315m = -2106\,;\quad C_6 = +24{\cdot}563w + 1{\cdot}044m = +8861.$

By substituting in equation (24) the forces and moments are as in (32).

$$\left.\begin{aligned}
&T_t = -1983 - 555\sin\theta - 961\cos\theta + 947\theta\,;\quad T_n = -755\cos\theta + 1306\sin\theta\,;\\
&T_b = 512\sin\theta + 886\cos\theta + 1027\theta - 2152\,;\\
&M_t = -6276 - 2106\sin\theta + 8861\cos\theta - 1492\,\theta\cos\theta + 2585\,\theta\sin\theta + 2997\theta\,;\\
&M_n = -4896\cos\theta - 8539\sin\theta + 2030\theta\sin\theta + 3518\theta\sin\theta + 4896\,;\\
&M_b = 4933\sin\theta - 2986\cos\theta + 1377\theta\cos\theta - 2383\,\theta\sin\theta - 2764\theta - 5797.
\end{aligned}\right\} \quad (32)$$

To check these results, from equations (31)

$$T_t(0) = -2944 \text{ lb.}\,;\quad T_t\left(4\frac{\pi}{3}\right) = +2944 \text{ lb.}\,;\quad T_n(0) = -755 \text{ lb.}\,;$$

$$T_n\left(4\frac{\pi}{3}\right) = -755 \text{ lb.}\,;\quad T_b(0) = -1266 \text{ lb.}\,;\quad T_b\left(4\frac{\pi}{3}\right) = +1266 \text{ lb.}\,;$$

$$M_t(0) = +2585 \text{ ft.-lb.}\,;\quad M_t\left(4\frac{\pi}{3}\right) = -2585 \text{ ft.-lb.}\,;\quad M_n(0) = 0\,;$$

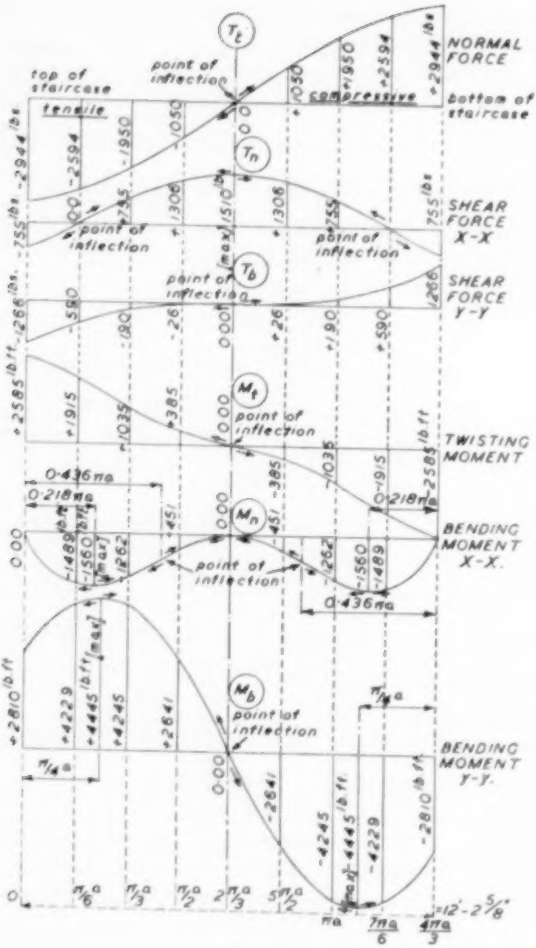

Fig. 9.

$$M_n\left(4\frac{\pi}{3}\right) = 0\,; \quad M_b(0) = +\,2810 \text{ ft.-lb.}\,; \quad M_b\left(4\frac{\pi}{3}\right) = -\,2810 \text{ ft.-lb.}$$

By differentiation the following are derived and give the maximum and minimum values.

$\dfrac{dT_t}{d\theta} = -\,555 \cos\theta + 961 \sin\theta + 947 = 0\,;$ as the roots are outside the limits there is no maximum or minimum value.

$\dfrac{dT_n}{d\theta} = 755 \sin\theta + 1306 \cos\theta = 0,$ for $\theta = \dfrac{2\pi}{3}$ then $T_{n\,\max.} = +\,1510$ lb.

$\frac{dT_b}{d\theta} = 512 \cos\theta - 886 \sin\theta + 1027 = 0$; as this has no roots T_b increases as θ increases.

$\frac{dM_t}{d\theta} = -3598 \cos\theta - 6276 \sin\theta + 1492\theta \sin\theta + 2585\theta \cos\theta + 2997 = 0$; as this has no roots M_t decreases as θ increases.

$\frac{dM_n}{d\theta} = 6926 \sin\theta - 5021 \cos\theta + 2030\theta \cos\theta - 3518\theta \sin\theta = 0$; for $\theta = 0{\cdot}218\pi$, $M_{n\,\text{min.}} = -1560$ ft.-lb.; for $\theta = 1{\cdot}115\pi$, $M_{n\,\text{min.}} = -1560$ ft.-lb.; for $\theta = \frac{2\pi}{3}$, $M_{n\,\text{max.}} = 0$.

$\frac{dM_b}{d\theta} = 6310 \cos\theta + 603 \sin\theta - 1377\theta \sin\theta - 2383\theta \cos\theta - 2764 = 0$

for $\theta = \frac{\pi}{4}$, $M_{b\,\text{max.}} = +4445$ ft.-lb.; for $\theta = \frac{13}{12}\pi$, $M_{b\,\text{min.}} = -4445$ ft.-lb.

The points of inflection are found from the second differentials of equations (32) and are

$\frac{d^2T_t}{d\theta^2} = 555 \sin\theta + 961 \cos\theta = 0$, that is when $\theta = \frac{2\pi}{3}$;

$\frac{d^2T_n}{d\theta^2} = 755 \cos\theta + 1306 \sin\theta = 0$, that is when $\theta = \frac{\pi}{6}$ and $\frac{7\pi}{6}$;

$\frac{d^2T_b}{d\theta^2} = -512 \sin\theta - 886 \cos\theta = 0$, that is when $\theta = \frac{2\pi}{3}$;

$\frac{d^2M_t}{d\theta^2} = 5090 \sin\theta - 3961 \cos\theta + 1492\theta \cos\theta - 2585\theta \sin\theta = 0$, that is when $\theta = \frac{2\pi}{3}$;

$\frac{d^2M_n}{d\theta^2} = 8596 \cos\theta - 1503 \sin\theta - 2030\theta \sin\theta - 3518\theta \cos\theta = 0$, that is when $\theta = 0{\cdot}436\pi$ and $\frac{4\pi}{3} - 0{\cdot}436\pi$;

$\frac{d^2M_n}{d\theta^2} = -7687 \sin\theta - 1780 \cos\theta - 1377\theta \cos\theta + 2383\theta \sin\theta = 0$, that is when $\theta = \frac{2\pi}{3}$;

The points of zero values are:

$T_t = 0$ when $\theta = \frac{2\pi}{3}$; $T_n = 0$ when $\theta = \frac{\pi}{6}$ and $\frac{7\pi}{6}$; $T_b = 0$ when $\theta = \frac{2\pi}{3}$;

$M_t = 0$ when $\theta = \frac{2\pi}{3}$; $M_n = 0$ when $\theta = 0, \frac{2\pi}{3}$ and $\frac{4\pi}{3}$; $M_b = 0$ when $\theta = \frac{2\pi}{3}$.

The diagrams of forces and moments are shown in *Fig.* 9. In accordance with the signs shown in *Figs.* 1 to 4, the normal forces are tensile at the top and compressive at the bottom of the staircase. The bending moment $x - x$ causes tension at the bottom face of the beam when it is negative, and the bending moment $y - y$ causes tension at the inside face of the beam when it is positive.

Book Reviews.

"Proceedings of a Symposium on Concrete Shell Roof Construction." (Obtainable from Concrete Publications, Ltd. Price 31s. by post; 6 dollars in Canada and U.S.A.)

THIS volume of 258 pages comprises the papers read at a symposium arranged by the Cement and Concrete Association and held in London in July 1952. The papers deal with a wide range of problems concerning the design and construction of shell roofs, including the architectural aspect, design, research, construction, shuttering, and so on. Engineers from several countries took part in the discussions that followed the papers, and full reports of these discussions are a valuable feature of the book. There are more than 180 photographs and diagrams.

The papers are as follows. "Domes, Vaults, and the Development of Shell Roofing", by Leo M. De Syllas; "Various Forms of Shell Roofing and their Application", by Edward D. Mills; "Architectural Problems of Shell Roofing", by E. Leslie Gale; "Existing Methods for the Analysis of Concrete Shell Roofs", by J. J. McNamee; "Flexibility Coefficient Methods and their Application to Shell Design", by A. Goldstein; "Research on Shells", by P. B. Morice; "The Theory of New Forms of Shell", by R. S. Jenkins; "The Combination of Shells and Prestressing", by C. V. Blumfield; "Design and Construction from the Economic Aspect", by H. G. Cousins; "Construction of Shelton Grange Power Station at Leeds and a Factory at King's Lynn", by H. E. Manning; "Formwork used on a Factory at Greenford", by H. F. Rosevear; "Construction of Self-supporting Reinforced Concrete Vaults at Antwerp", by A. Paduart and C. Wets.

"Reinforced Concrete Arch Design." By G. P. Manning. (London: Sir Isaac Pitman & Sons, Ltd. 1954. Price £2 10s.)

THIS book gives all the information necessary for the design of arch bridges in reinforced concrete. It is clearly written, and the reader should have little difficulty in applying the methods successfully in the design of bridges, or indeed arch ribs for any structure. Fixed, pinned, and tied arches, with solid and piled abutments, are considered, together with a chapter on continuous arches on flexible supports which are treated by the author's "displacement method of frame analysis". The clarity of the graphs and drawings deserves mentioning. This is the second edition of a book published in 1932, and a minor criticism is that in the chapter on temporary hinges the reader is referred to a book long since out of print, while there are no references to more recent information on this subject.—J. E. G.

"Theory of Structures." By H. W. Coultas. (London: Sir Isaac Pitman & Sons, Ltd. Price 25s.)

THIS is the fourth edition of a work first published in 1925, and which is intended for the use of students in obtaining a degree in engineering or membership of a professional society. In this edition, comprising 556 pages, a chapter is included on prestressed concrete.

"Building Technicians' Diary 1955." (London: Association of Building Technicians. Price 5s. 4d.)

THIS pocket-size diary contains 126 pages of data for everyday reference relating to building. In addition to the diary pages there are maps of Great Britain, some graph pages, and blank pages.

A Multiple-story Precast Building in Riga.

In the Russian periodical "Stroitelnaya Promishlennost" for November, 1954, some details are given of a multiple-story precast building being erected in Riga. The weight of the precast members was limited to 1½ tons by the lifting capacity of the crane. Since some of the columns are to carry very heavy loads and measure 20 in. by 20 in. in cross section, solid columns, which would weigh about 2½ tons, could not be used. Hollow columns of story height are therefore being used and filled with in-situ concrete after the precast beams are placed in position, thereby obtaining a monolithic connection between the columns and the beams (*Fig.* 1).

At first the columns were cast on the site, but later they were made in a factory. The columns cast on the site were concreted in a vertical position, the hole being formed by a cast-iron tube. The tube was filled with damp sand, and vibrated as it was lifted up out of the mould as concrete was placed between the mould and the tube. The use of sand as a temporary filler was found to be satisfactory.

The method of assembling reinforcement for the columns is shown in *Figs.* 2 and 3. Each group of four bars, with short lengths of steel pipe as spacers, was assembled separately and then fixed in steel templates while the links were welded. The links are round bars and steel straps placed alternately. Four small cleats with bolts (*Fig.* 4) were used

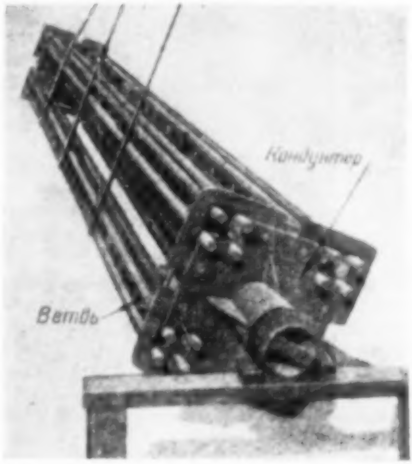

Fig. 2.—Jig for Assembling Bars.

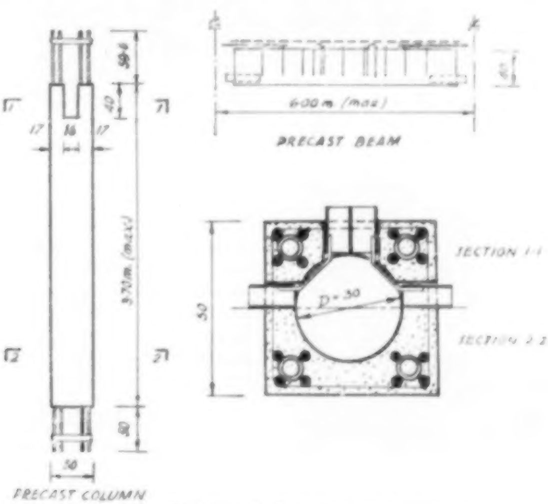

Fig. 1.—Details of Column and Beam.

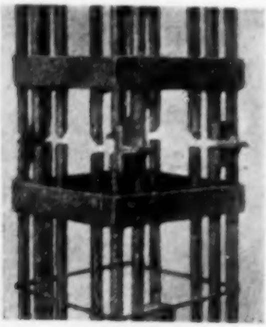

Fig. 3.

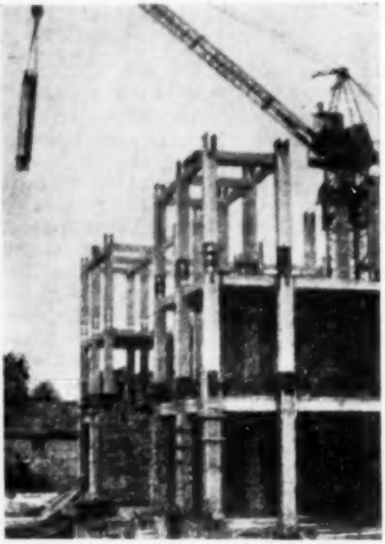

Fig. 4.—Lifting a Column.

to secure the columns in position before the bars were welded. The erection, alignment, and fixing of 44 columns for a story requires 12 men-days; the placing of 72 precast beams per story, 9 men-days; welding the beam reinforcement in the column joints, 6 men-days; welding the column reinforcement in the column joints, 22 men-days; ancillary work, 30 men-days.

Centrally-mixed Concrete.

THE illustration shows the type of tipping lorry, with a boat-shaped body, used for transporting concrete over a 10-mile stretch of new road, including 14 bridges, in Ohio, U.S.A. The total amount of concrete transported in this way was 95,000 cu. yd.

The mixing plant was at about the middle of the length of the road, and two concrete mixers discharged into a hopper of 4 cu. yd. capacity, which in turn was discharged into the lorries whose bodies also were of 4 cu. yd. capacity. Fourteen of these lorries were used, and travelled the maximum distance of five miles in eighteen minutes. The average time for loading them was $1\frac{3}{4}$ minutes, and of discharging them 1 minute. The average rate of progress in laying the road slab was 2500 linear feet of road 12 ft. wide in an 8-hour day. [This note is abstracted from "Engineering News-Record," February 10, 1955.]

Concreting in Cold Weather.

A SYMPOSIUM on Winter Concreting is to be held in Copenhagen in February 1956 under the auspices of Réunion Internationale des Laboratoires et de Recherches sur les Materiaux et les Constructions. A brochure, printed in the English language, giving full details of the arrangements, is obtainable from the Director of Research, The Danish National Institute of Building Research, Burgergade 20, Copenhagen, Denmark.

Cooling Concrete Aggregates.

In the construction of a dam at Vaitarna, India, the aggregates were cooled before the materials were proportioned. The work is described in "Beton og Jernbeton" for January, 1954, by Mr. C. S. Forum, of Messrs Christiani & Neilsen. The following notes are abstracted from this article.

The dam is to be 1820 ft. long with a maximum height of 250 ft., and will contain 20,000,000 cu. ft. of concrete, most of which is a lean mixture using aggregates of 6 in. maximum size. The aggregates are basalt quarried about three miles from the site, where it is crushed and screened into six sizes, namely from 6 in. to 3 in., 3 in. to 1½ in., 1½ in. to ¾ in., ¾ in. to ⅜ in., ⅜ in. to ⅛ in., and smaller than ⅛ in. The particles are angular and irregular in shape. From the storage silos at the screening plant the aggregates are transported by an aerial ropeway to stockpiles at the site of the dam. From the stockpiles they are taken to the batching plant through a cooling tunnel.

The specification requires that the concrete should have a temperature of not more than 60 deg. F. when it is placed, and the aggregates are cooled by cold water and cold air. This is done by carrying the different sizes of aggregates on separate conveyor belts through an insulated cooling tunnel about 300 ft. long, where the four larger sizes are sprayed (*Fig.* 1) with water, cooled to about 35 deg. F., while the two smallest sizes are subjected to an air-stream with a temperature of about 40 deg. F. The aggregates pass through the tunnel in 20 to 25 minutes, during which time the coarse aggregates are cooled from about 105 deg. F. to about 40 deg. F., and the fine material to about 60 deg. F. The automatic weigh-batching plant (*Fig.* 2) has three mixers of 2 cu. yd. capacity and a total capacity of 120 cu. yd. of concrete per hour.

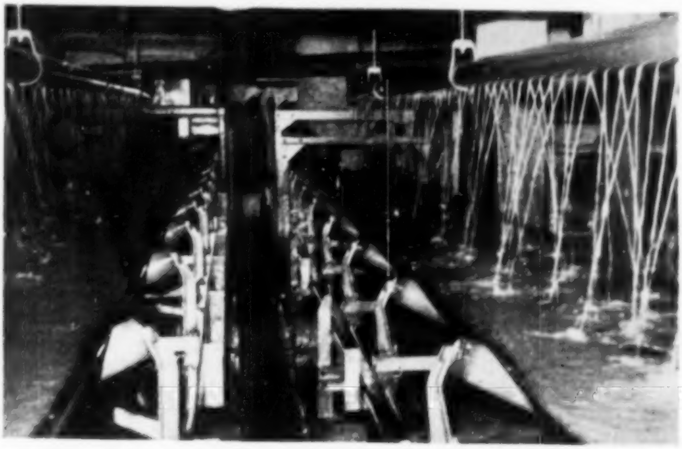

Fig 1.—Method of Cooling Aggregates.

The concrete has a slump of about ¾ in. and a water-cement ratio of 0·64 to 0·66. Test cylinders are made three times in every 8-hours' shift. The cement content is 345 lb. per cubic yard and the average strength is 4500 lb. per square inch.

Two cableways, each with a working load of ten tons and a span of 2100 ft., are used to transport the mixed concrete in pneumatically-operated skips containing 4 cu. yd. The concrete is compacted by large vibrators operated by two men. The dam is being built in 36 sections each 50 ft. wide. The concrete is placed in

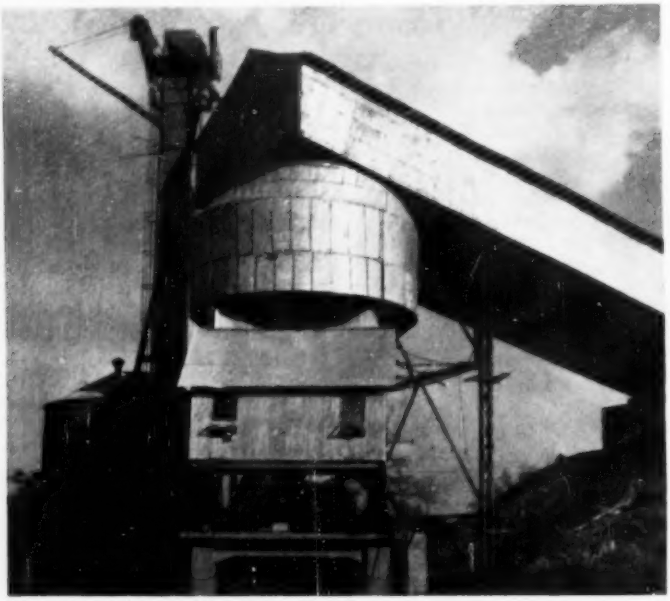

Fig. 2.—Mixing Plant.

lifts of 5 ft., each of these lifts being placed in three or four layers in steps from the downstream face to the upstream face of the dam in order to reduce shrinkage effects, and to improve the bond between the layers. The stiff consistency enables the workmen to walk on the fresh concrete, and with the use of two heavy vibrators the 4 cu. yd. delivered by each skip are compacted in an average of 2½ minutes.

The dam was designed by the Bombay Minicipality and is being built for them by the Hindustan Construction Co., of Bombay. Messrs. Christiani and Nielsen are the consulting engineers to the contractors for the work at the site.

Foamed Cement as Insulation.

A METHOD of producing foamed cement for insulation purposes is described in "Engineering News-Record" for February 24, 1955. Foam is generated at the rate of 60 to 120 gallons per minute by aerating by compressed air, in a tank, a compound with a protein base. Cement and water are stirred to form a slurry in a concrete mixer with a drum of 5 cu. yd. capacity; to this the foam is added through a pipe, and the rotation of the mixer drum is continued in order to distribute the air cells throughout the slurry. The foamed slurry is discharged from the mixer into a hopper from which it is taken to the site of the work by means of concrete carts.

In the work described, the material was used to cover a roof of 60,000 sq. ft. The density was 40 lb. per cubic foot, and it is stated that this can be varied between 10 lb. and 100 lb. per cubic foot according to the proportion of cement and foam in the mixture. At a density of 40 lb. per cubic foot, about 575 lb. of cement per cubic yard of foamed material and about 105 gallons of foam were used per cubic yard. The foam cost about 5s. per cubic yard of concrete. The process is owned by National Foam System, Inc.

An Experimental Prestressed Road in France.

CONCRETE COMPRESSED WITHOUT STEEL.

About 325 yd. of the road between Bourg and Lyon, France, has been replaced by a concrete slab prestressed longitudinally, without the use of steel bars or wires, between fixed abutments. Parts of the slab are reinforced transversely with mild-steel bars and parts are prestressed by cables. The transverse compressive stress extra width was consolidated by a sheeps'-foot roller, after which sand and gravel were laid and thoroughly compacted over the whole width of the new carriageway. The new slab is 23 ft. wide by 4¾ in. thick and was cast on waterproof paper laid on the prepared surface. The edges of the road are formed by precast kerbs in which

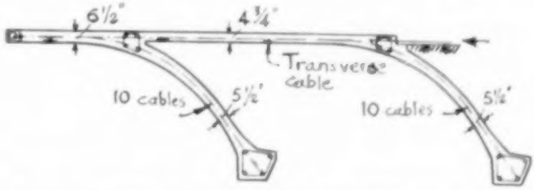

Fig. 1.—Section Through Abutment to Resist Thrust by Weight of Soil.

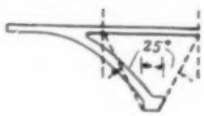

Fig. 2.—Section Through Abutment.

[Showing the volume of soil effective in resisting thrusts.]

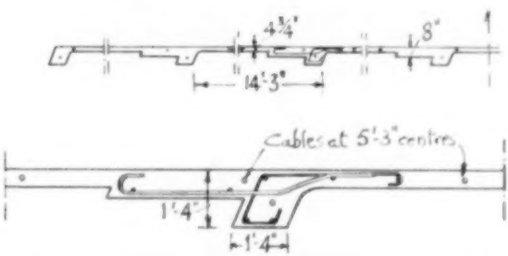

Fig. 3.—Section Through Abutment Resisting Thrust by Passive Resistance of the Soil.

is varied in different parts by altering the spacing of the cables. The design and construction of the road are described by M. H. Dollet and M. M. Robin in a recent number of the French journal "Travaux."

The sub-soil comprises glacial mud containing more than 70 per cent. of particles less than 0·016 in. diameter. The original macadam road was badly damaged, and the new slab is about 4 ft. wider. The the anchors for the transverse cables were cast; the kerbs also formed a track for the machines used to place and shape the bituminous surface.

The Abutments.

The longitudinal compression was produced by "flat-jacks", placed in transverse joints formed at intervals of about 195 ft., compressing the concrete against abutments at each end of the road. The

abutments are of two types, one (*Figs.* 1 and 2) relying for its stability on the weight of the earth contained between the dotted lines in *Fig.* 2, and the other (*Fig.* 3) utilizing the passive pressure of the earth to prevent horizontal movement.

The centre-line of the curved part of the abutment shown in *Figs.* 1 and 2 is a logarithmic spiral. It can be shown that a horizontal force, acting in the direction of the arrow and tending to cause a horizontal displacement of the abutment, is transformed into a vertical force tending to cause a vertical displacement by reason of the curved slabs sliding over the shearing plane of the ground. For example, if the angle of friction between the concrete and the soil were 35 deg. the vertical force would be about one-third of the horizontal force, and the resistance to this vertical force would be provided by the soil contained between the dotted lines (*Fig.* 2) representing the angle of internal friction of the soil.

This type of abutment has been used for several important works, notably the airfield at Orly. In the case of the experimental road, however, the conditions were not so severe as in most works because the ends of the prestressed slab abut existing road slabs which themselves are very stiff and also protect the soil near the surface. For these conditions an abutment (*Fig.* 3) was developed which relies on the passive pressure of the earth in front of the downwardly-projecting inclined keys. Several shallow keys were used, rather than one deep key, in order to reduce the bending moments on the slab, and in order to utilise the full passive resistance of the earth it was necessary to space the keys widely apart.

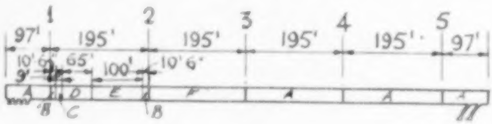

Fig. 4.—Longitudinal Section of Road.

A, 1 cable every 5 ft. 3 in. B, 2 cables. C, 8-mm. (0·3 in.) bars at 3 ft. 3 in. centres. D, 8-mm. bars at 1 ft. 8 in. centres. E, 16-mm. bars at 18 ft. 8 in. centres. F, 1 cable every 3 ft. 3 in.

Longitudinal and Transverse Prestressing.

The longitudinal compression was obtained by "flat-jacks" placed in joints numbered 1 to 5 in *Fig.* 4. Eighteen jacks were used in each joint. The jacks were placed in pairs, and between each pair was a wedge-shaped precast block which served to space the jacks and connect the two parts of the road. The jacks were grouted in the joints when the slabs were compressed.

The jacks were inflated and were left in the joints so that the compressive forces could be adjusted. The final compressive stress, after allowing for all losses, was not less than 215 lb. per square inch. Transversely the slab was either prestressed by cables comprising twelve 5-mm. wires or reinforced with mild steel bars as indicated in *Fig.* 4. Transverse compression is considered to be essential as cracks have been observed in Parts C, D, and E (*Fig.* 4), while in part F, which contains cables at 10-ft. 9-in. centres only, no cracks have yet been seen. There is not, as yet, sufficient information to determine the least compression required. The cracks occurred at the edge of the old road underlying the new slab, and their absence in the parts transversely prestressed indicates the value of transverse prestressing of slabs on foundations of varying stiffness.

The concrete was a 1 : 2 : 4 mixture with rapid-hardening Portland cement; cube crushing strengths at an age of four days were between 4150 lb. and 6350 lb. per square inch. The percentage costs per unit area of the road were: Concrete materials, mixing, and placing, 55; Longitudinal prestressing, 13; Transverse prestressing, 18; Precast kerbs, 14.

The road was designed by the Department des Ponts et Chaussées in collaboration with M. E. Freyssinet and the Société Technique pour l'Utilization de la Précontrainte. The work was carried out by Entreprise Maillard et Duclos.

Design of Statically-determinate Beams and Slabs in Prestressed Concrete based on Ultimate Load.

By P. W. ABELES, D.Sc., M.I.Struct.E.

This paper was prepared as the British contribution to the First Congress of the International Federation of Prestressing held in London in October 1953, and its contents are summarised in the General Report by Professor G. Magnel. Additional remarks and up-to-date references are added as footnotes.

WITH prestressed concrete the permissible stresses under working load have no relationship to those at failure. It is therefore necessary to investigate also the conditions at ultimate load.* In the following, the required magnitude of the factor of safety is not discussed, since this cannot be settled for prestressed concrete separately, but must be considered for each type of structure and applies to all materials.

There are different views on the state at which conditions at failure have to be considered. According to the "First Report on Prestressed Concrete" of 1951 [1], failure relates to the maximum load that can be supported in a single static loading. In Germany, Professor Ruesch advocates [2] that failure should be based on sustained loading. This suggestion, however, does not correspond with general practice.

In the "First Report" [1] it is stated that failure of beams in bending may be due to one or more of the following causes :

(1) Fracture of the steel in tension.

(2) Excessive elongation of the steel followed by crushing of the concrete.

(3) Crushing of the concrete without substantial elongation of the steel.

(4) Fracture of the concrete due to shear.

(5) Failure of the anchorage or by slipping due to limited bond between the steel and the concrete.

It is further stated that causes (3), (4), and (5) should be avoided, as failure is sudden. In the following, first the question of bending failure is investigated, corresponding to causes (1) to (3), and afterwards failure due to shear [cause (4)], and failure due to bond-slip [cause (5)] is briefly discussed, as well as the question of fatigue.

In reinforced concrete failure occurs either when the maximum possible elongation of the steel is reached (under-reinforced beams) or when the section fails primarily in compression (over-reinforced beams). In both cases the maximum possible strain is reached in the compressive zone of the concrete at failure ; in under-reinforced beams, however, the properties of the concrete influence only slightly the ultimate load, while with over-reinforced beams those properties alone govern failure. If mild steel is used in under-reinforced beams, the maximum possible elongation in the steel, which may vary with different conditions (for example, the percentage of the reinforcement and the plasticity and strength of the concrete) corresponds always to the yield-point stress, and consequently there is no difficulty in determining the maximum force that can be resisted by the reinforcement. Thus, a so-called "balanced" design is obtained with the percentage of reinforcement at which the maximum compressive resistance, that is the maximum compressive force that can be resisted by the section, equals the maximum possible tensile resistance, all higher percentages in which the compressive resistance is less being called over-reinforced.

In the case of prestressed concrete two

* The necessity for such investigations was indicated by the writer in "Concrete and Constructional Engineering" in February, 1941, and in his contribution to Mr. T. J. Gueritte's paper, "Further Data concerning Prestressed Concrete" (Jour. Inst. Civ. Engrs., No. 8, 1941), in which the writer investigated the formula $t_{ult.} = \frac{M_{ult.}}{A_t\, a_1\, d}$ by various test results.

cases must be considered, namely, with bonded and with non-bonded steel. According to the author's knowledge, Professor R. H. Evans [3] was the first, in 1942, to draw attention to the different behaviour of prestressed concrete with bonded wire and non-bonded bars. Professor A. L. L. Baker [4] noticed, in tests at Imperial College, a similar difference in the behaviour of non-bonded post-tensioned cables and emphasized the significance of the reduced ultimate resistance. Non-bonded tensioned cables will never fracture at failure, since such cables will elongate not only at, and adjacent to, the cracks but over their total length, and thus the maximum possible elongation in the steel related to the maximum compressive strain in the concrete will be reached much earlier. However, if part of the steel is bonded, whether it is tensioned or not the conditions of cracking are greatly improved, and higher ultimate strengths are reached, as for example tests of the American Navy have shown (exhibited at the Conference on Prestressed Concrete in Los Angeles in November 1952).

There is some similarity between prestressed and reinforced concrete at failure, but in comparing the two cases of bonded and non-bonded tensioned steel different types of reinforced concrete have to be considered. If ordinary mild steel bars of large diameter are used, the bond between the concrete and the steel is entirely destroyed between the cracks, and the assumptions made in designing reinforced concrete apply (that is the co-operation of the concrete in compression and the steel in tension and no tensile resistance of the concrete). Such a reinforced concrete beam acts at failure like an arch (*Fig.* 1). The behaviour of prestressed concrete with non-bonded steel is similar (*Fig.* 2), few and wide cracks developing in the beam. However, there is a difference in assessing the load that will cause failure.

Non-bonded tensioned steel, whether wires, cables, or high alloy steel bars, have no distinct yield point, and consequently the maximum possible elongation in an under-reinforced section is not known. It depends on the percentage of steel and the properties of the materials, and also governs the ultimate-load conditions. Thus, the corresponding ultimate force in

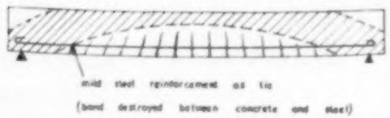

Fig. 1.—Under-reinforced Beam approaching Failure.

Fig. 2.—Beam with Non-bonded Steel approaching Failure.

the steel may vary within wide limits, depending on the initial prestress, on the shape of the stress-strain diagram of the steel, and on the strength and plasticity of the concrete.

With well-bonded pre-tensioned steel, the behaviour at failure is very similar to that of reinforced concrete in which the reinforcement has no distinct yield point. This type of reinforced concrete has different properties from concrete reinforced with steel having a distinct yield point. Instead of few and wide cracks, many fine cracks develop, and the bond is destroyed only in the immediate neighbourhood of the cracks if concrete of high strength is used. The stress in such untensioned steel will reach, in most cases, the ultimate strength of the steel as shown in various publications based on test results [5–10]. Unfortunately, it has not yet been possible to measure the maximum strain in a crack or the corresponding stress. Consequently, all assumptions for the actual maximum possible elongation of the steel for different percentages and materials are based on measurements of strain extending over cracked and uncracked portions of the tensile zone, and are therefore not conclusive. Therefore, a design based on the physical conditions at failure, that is the maximum elongation of the steel and the maximum shortening of the concrete, is in the present state of knowledge not possible, at least for bonded steel.

It seems, therefore, advisable to employ a simple method of approximation that gives safe values. There is hardly any difference between the results of the various methods of design of under-reinforced sections, since the lever-arm differs only little for different stress diagrams of

concrete,* and the maximum tensile force is, in most cases, based on the ultimate strength of the steel. It is therefore unnecessary to employ complicated formulæ in the design of under-reinforced sections, but (except for sections with a relatively shallow top flange) it is not advisable to over-simplify the design to such an extent that a constant depth-ratio is taken into account as the lever-arm independently of the percentage of reinforcement and the strength of the concrete.† A difference in the results of using different design formulæ occurs only when the so-called "balanced" design is reached and over-reinforced sections are considered. Professor R. H. Evans has shown [11] that Whitney's ultimate-load formula based on plastic design for reinforced concrete [12] applies also satisfactorily to prestressed concrete.

The writer has presented a slightly modified formula [13], and many test results have shown that it is safe for constructions with well-bonded steel independently of whether it is tensioned or not. *Fig.* 3 shows the simplified stress diagrams for under-reinforced and for "balanced" design on which the formulæ are based **. A comparison between the formula and many test results proves that the members tested exceed the safe strengths assumed if the steel is bonded, while with non-bonded steel the test results are considerably lower [13]. *Fig.* 4 is a design diagram for steel with an ultimate strength of 224,000 lb. per square inch, assuming that the factor of safety is two. This diagram can be used also for steels with different strengths, as shown. The average ultimate stress in the concrete assumed approximates to the strength of prisms, which is generally two-thirds to three-quarters of the strength of cubes; if no data are available, 60 per cent. of the cube strength can be considered safe.

A greater factor of safety is often required for concrete than for steel (for example $2\frac{1}{2}$ for concrete and 2 for steel). In this case, the ultimate stress in the concrete must be reduced accordingly, (for example $\frac{2}{2{\cdot}5} = 0{\cdot}8$ times its value) if the formulæ in *Fig.* 3 are used. If also the compressive zone is under initial compression, this must be allowed for by a reduction of c_m. It is obvious that the formulæ in *Fig.* 3 are only an approximation, as can be seen from *Fig.* 4, since it may be expected that there is no direct break in the graphs at the percentages in the "balanced" design, nor is the carrying capacity of all over-reinforced sections the same. It may be expected that a gradual slight increase occurs from the point of "balanced" design with increasing percentages of steel, and that even slight differences may have to be considered in the magnitude of the percentage for the "balanced" design, depending on the magnitude of the effective prestress. Professor A. L. L. Baker [4, 14] has suggested a design based on the maximum strains of the materials in accordance with their stress-strain properties, and on the influence of the prestress. However, as already mentioned, strain measurements cannot be satisfactorily obtained for high-tensile steel well bonded to the concrete, and the percentage of steel in a "balanced" design depends on the correct assessment of the maximum strain in the steel; reference may be made to the discussions between Professor A. L. L. Baker and others and Dr. K. Hajnal-Konyi [10]. The principal idea of Professor Baker's argument, namely that the carrying capacity will vary with the magnitude of the prestress, seems to be correct, although the differences between the lower and higher prestress might have much less influence on the ultimate load than is envisaged by him. It will require extensive research to ascertain this, and tests are being carried out at the University of Illinois ‡.

With regard to grouted post-tensioned steel, it depends entirely on the efficiency

* It should be understood that this relates only to the shape of the stress diagrams and not to the strength of the concrete, and should not be considered as a statement that the strength of the concrete is of so little influence that it can be ignored. The latter would result only in a gross approximation.

† Such an over-simplification is embodied, for example, in the French Draft Code of Practice for Prestressed Concrete of 26.10.1953, where a constant lever arm of 0·9*d* is taken into account for all under-reinforced beams independent of the percentage of steel, the strength of the concrete, and the efficiency of the bond of the tensioned steel. (See, e.g., "Cours de Béton Précontraint," by J. R. Robinson. Paris: Dunod, 1954, p. 81.)

** Such a stress diagram for under-reinforced beams was shown by the writer in "Concrete and Constructional Engineering" for June 1946 (Fig. 4), May 1947 (Fig. 9), and April 1948 (Fig. 1*d*), and for balanced design was shown in "Concrete and Constructional Engineering" for October 1951 (Fig. 4).

‡ A report on the tests was published in the Journal of the American Concrete Institute, June 1954, as Paper No. 50-49. However, the results relate only to post-tensioned wires with limited bond efficiency.

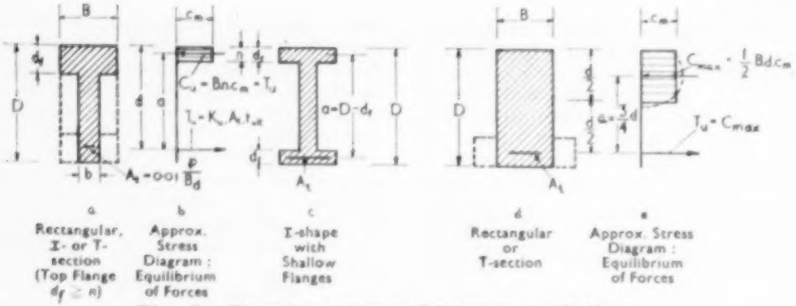

Fig. 3.—Simplified Stress Diagrams at Failure.

UNDER-REINFORCED SECTIONS.
(a, b, and c).

From $C_u = T_u$:

$$n = \frac{K_u A_t . t_{ult.}}{B . c_m} = 2vpd.$$

$$a = d - \frac{n}{2} = d(1 - vp),$$

where $v = \frac{K_u . t_{ult.}}{200 c_m}$.

$$M_u = T_u a = K_u A_t . t_{ult} d(1 - vp) = 2vp(1 - vp) . B . d^2 . c_m$$

or : $$d = \sqrt{\frac{1}{2vp(1 - vp)c_m}}\sqrt{\frac{M_u}{B}}.$$

For cross section (c) :

$$M_u = K_u A_t . t_{ult.}(D - d_f). \; [d_f = 2vpd].$$

OVER-REINFORCED SECTIONS.
(d and e).

$$M_{max.} = \frac{3}{8} Bd^2 . c_m = 0{\cdot}375 Bd^2 . c_m$$

or : $$d = \sqrt{\frac{1}{0{\cdot}375 c_m}}\sqrt{\frac{M_u}{B}}.$$

Safe value for $c_m = 0{\cdot}6 c_u$:

$$M_{max.} = 0{\cdot}225 Bd^2 c_u$$

or $$d = \sqrt{\frac{1}{0{\cdot}225 c_u}}\sqrt{\frac{M_u}{B}}.$$

NOTATION

$M_u = FS . M_w$ = bending moment at ultimate load (FS = factor of safety. M_w = bending moment at working load).

$t_{ult.}$ = ultimate strength of the steel.

K_u = reduction factor, depending on efficiency of bond ($K_u = 1$ for fully-efficient bond ; $K_u = 0{\cdot}7$ if there is no bond and the initial prestress is $\frac{2}{3}$ of $t_{ult.}$).

c_u = strength of concrete cubes.

c_m = maximum average stress in concrete under ultimate load conditions in a rectangular stress-block (safe value of $c_m = 0{\cdot}6 c_u$).

$A_t = A_{ts} + A_{tu}$ (the entire tensile reinforcement A_t may be composed of tensioned reinforcement A_{ts} and untensioned reinforcement A_{tu}).*

B = width of top flange.

b = width of web.

D = entire depth.

d = depth related to the centroid of A_t.

d_f = depth of top flange (*Fig.* 3a).

n = depth of approx. rectangular stress block in an under-reinforced beam.

a = lever-arm between the ultimate tensile and compressive forces.

$p = \frac{100 A_t}{Bd}$ = percentage of steel.

$v = \frac{K_u t_{ult.}}{200 c_m}$ = stress ratio.

$T_u = K_u A_t . t_{ult.}$ = ultimate tensile force developed in an under-reinforced or a " balanced " beam.

$C_u = Bnc_m$ = ultimate compressive force in an approximately rectangular stress-block resisted by the concrete in an under-reinforced section.

$C_{max.} = \frac{1}{2} Bdc_m$ = safe maximum ultimate compressive force in a " balanced " or an over-reinforced beam.

* The untensioned steel fully co-operates to its ultimate strength if it is well-bonded (e.g. pairs of slightly twisted wires) and A_{tu} does not exceed A_{ts}.

of the grouting whether beams will behave like those with pre-tensioned bonded steel or like those with post-tensioned non-bonded steel. Consequently it is necessary to consider a factor K_u in the formula for ultimate load conditions (*Fig.* 3), which will be unity with fully-efficient bond but which may be 0·7 (or even less) if the bond is not efficient. The efficiency of the bond depends entirely on the design and execution. The writer knows of two cases relating to the same type of cable in which different results were obtained. In the first case the cable was placed in a plastic sheathing that did not allow the grout to be well injected, nor did it ensure bond to the concrete, while in the second case sufficient space was available for bonding. The corresponding values of K_u were 0·7 and 1.

There is a great difference in the behaviour of under-reinforced and over-reinforced beams; while the former fail after sufficient warning by extensive deformation and cracking, over-reinforced beams fail suddenly without appreciable deformation. Similarly to over-reinforced beams, sudden failure may also occur in under-reinforced members if the percentage of steel is so small that at the development of the first crack the tensile reinforcement is too weak to resist the entire tensile force. Obviously, a minimum percentage is required to avoid such conditions. In under-reinforced beams the steel may fracture in some cases at failure, although only after previous considerable deformation has given ample warning.

Failure due to shear in prestressed beams is particularly sudden, and it is therefore essential to make sure that under ultimate-bending conditions failure due to shear cannot occur. *Fig.* 5 shows a diagram taken from a paper prepared by the writer in 1951 [16], in which nominal

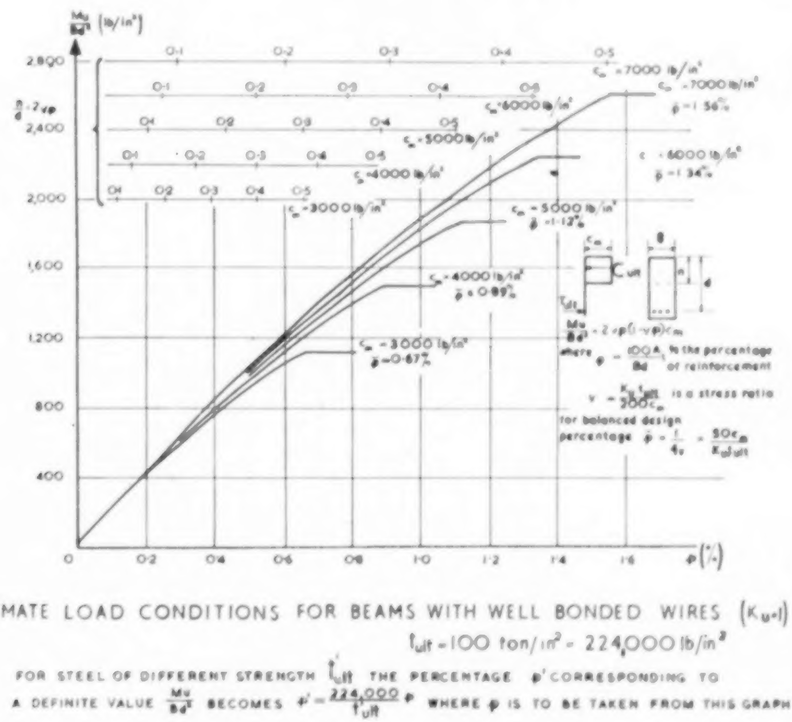

Fig. 4.—Graph according to Stresses in Fig. 3.

bending stresses at the bottom of a prestressed beam are plotted for a straight-line stress distribution, the beam being loaded by a central concentrated load. It is seen that under increased load large cracks develop in the central portion, where the nominal tensile stresses considerably exceed the modulus of rupture. In view of the shearing stresses, also plotted, it is clear that failure due to shear would take place near the centre where the shearing stress and the nominal bending-tensile stress coincide, and considerable cracking has already taken place. Therefore stirrups should be provided in the central portion as indicated in *Fig*. 5, these stirrups being calculated in the same way as in reinforced concrete but for ultimate load.

Tests have shown that relatively high shearing stresses under conditions at failure are not dangerous so long as the nominal tensile stresses under conditions at failure are low, but the combination of high nominal bending-tensile stresses with medium or high shearing stresses or of high shearing stresses with medium or high tensile stresses, requires the provision of stirrups or the thickening of the web of a beam. Insufficient test data are available to give definite values for design purposes. On the one hand failure due to shear must be avoided in any circumstances, for example by the provision of stirrups ; on the other hand it is advisable to limit the number of stirrups to the minimum possible so as to allow the use of internal vibrators and thus improve the

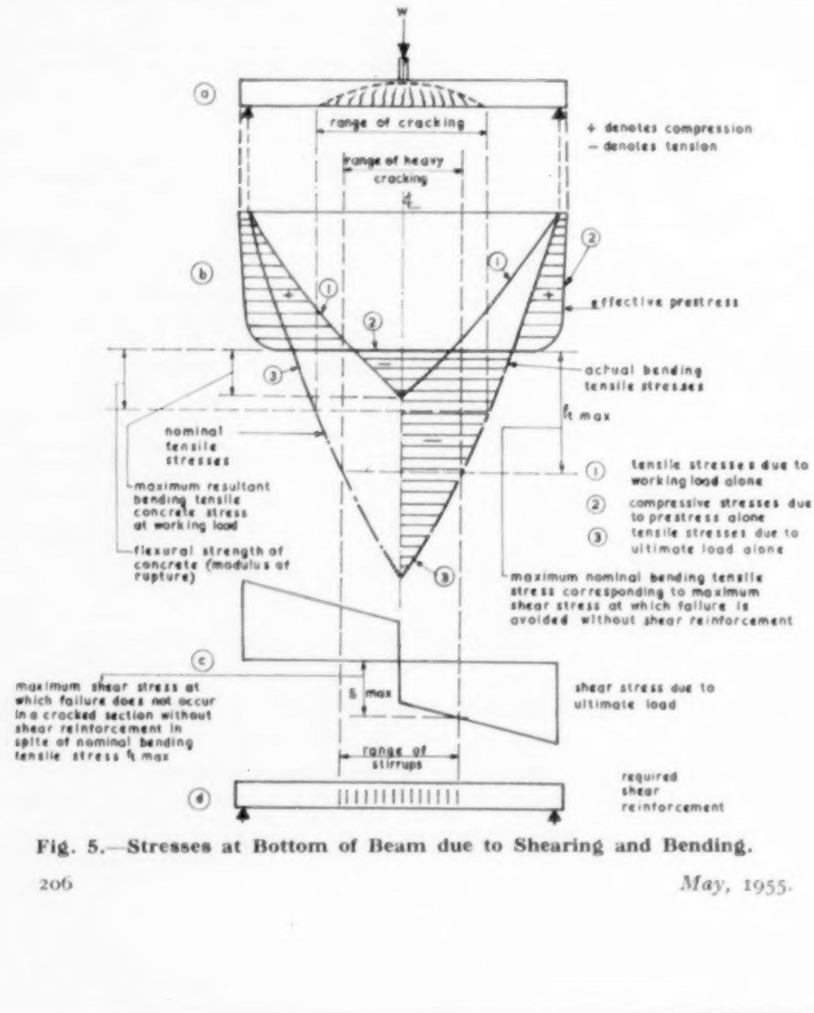

Fig. 5.—Stresses at Bottom of Beam due to Shearing and Bending.

quality of the concrete. Tests on failure due to shear are being made at the University of Illinois.* If an investigation for ultimate-load conditions indicates that stirrups are not required, a special design for conditions of shear at working load is superfluous.

In order to avoid slipping of pre-tensioned steel by bond, it was originally considered necessary to use only piano wire or wire of small diameter with an indented surface. However, tests and experience have shown that bond depends mainly on the surface condition of cold-drawn wire and less on the diameter, and particularly wire which had special treatment after being drawn has proved very satisfactory with a diameter as large as 0·2 in. or even 0·276 in.†, as tests by British Railways [17] have shown. No slipping need be considered in this case, while with much thinner wire without special treatment slipping may occur due to its smooth surface.

It does not seem necessary to investigate the fatigue-strength of prestressed concrete, that is the maximum loading of which 1,000,000 repetitions will be just resisted by the structure without failure.** It is more important to know whether repetition of loading under working-load conditions, or slightly increased loading, influences the factor of safety against static failure. Tests carried out for British Railways [18] have shown that in bridge-slabs with well-bonded wires of 0·2 in. diameter (60 per cent. tensioned and 40 per cent. untensioned) no reduction of the ultimate load calculated for $K_u = 1$ occurred after the slabs had previously been loaded until cracks developed and then subjected to a fatigue-loading at which the cracks opened and closed millions of times under the working load and increased loading. With non-bonded wires the resistance to fatigue of the steel alone governs the ultimate-load conditions, and consequently it is essential to avoid too high a stress in the steel at tensioning and under working load in order to avoid early failure due to fatigue.

Summing up, it may be stated that if beams or slabs are designed in accordance with the suggestions shown in *Figs.* 3 and 5 a simple and safe design to resist bending is obtained; but this relates only to members with bonded wires, and further investigations are necessary for members with non-bonded steel. It is of paramount importance to prevent failure due to shear, but further research is required to clarify also this question.

* A report on this test has appeared in the Journal of the American Concrete Institute, October 1954.

† Very satisfactory results were obtained by pre-tensioning a stranded cable of 1¼-in. diameter in the U.S. This cable, which had a tensile resistance of 31,000 lb. (ultimate strength about 240,000 lb. per square inch) was tensioned to a stress of 140,000 lb. per square inch and the prestressing force was transferred over a length of 6 in. only.

** Such an investigation was carried out on beams containing pre-tensioned smooth wires of 0·276 in. diameter by the Chief Civil Engineer's Department, Eastern Region, and the Research Department of British Railways in 1954, when nearly 10 million repetitions of loading were applied; in this case a fatigue failure load of about two-thirds of the static-failure load was obtained. At the latter load the maximum resistance of the material was reached and the wires fractured.

References.

(1) First Report on Prestressed Concrete. Inst. Struct. Engrs. London, 1951.

(2) Der Einfluss der Zeit auf Festigkeit und Verformung (The Influence of Time on Strength and Deformation), by H. Ruesch. Assoc. for Bridge and Struct. Eng., Fourth Congress, Final Report 1953.

(3) Relative Merits of Wire and Bar Reinforcement in Prestressed Concrete Beams, by R. H. Evans. Jour. Inst. Civ. Eng., Feb. 1942.

(4) A Plastic Theory of Design for Ordinary Reinforced and Prestressed Concrete including Moment Re-distribution in Continuous Members, by A. L. L. Baker. Mag. Conc. Research, 1949, Vol. 1, No. 2.

(5) Versuche mit Rechteckbalken, bewehrt mit besonders hochwertigem Stahl (Tests on Rectangular Beams Reinforced with Steel of Specially High Strength), by P. W. Abeles. Beton u. Eisen, Nos. 17 and 18, 1937.

(6) Further Data Concerning Prestressed Concrete, by T. J. Gueritte, contribution by P. W. Abeles. Jour. Inst. Civ. Eng., Oct. 1941.

(7) Fully and Partly Prestressed Reinforced Concrete, by P. W. Abeles, Jour. Am. Conc. Inst., Jan., 1945.

(8) Tests on Square Twisted Steel Bars and their Application as Reinforcement in Concrete, by K. Hajnal-Konyi. Structural Engineer, Sept. 1943; discussion Feb. and March 1944.

(9) Comparative Tests on Various Types of Bars of Reinforcement of Concrete Beams, by K. Hajnal-Konyi. Structural Engineer, May 1951; discussion January 1952.

(10) Tests on Concrete Beams reinforced with 12-Gauge Wires of an Ultimate Strength of 120 tons per sq. in., by K. Hajnal-Konyi. Mag. of Conc. Research, No. 9, March 1952.

(11) Research and Developments in Prestressing, by R. H. Evans. Unwin Lecture 1950. Jour. Inst. Civ. Eng., Feb. 1951.

(12) Plastic Theory of Reinforced Concrete Design, by Charles S. Whitney. Trans. Am. Soc. Civ. Eng., Paper 2133, 1940.

(13) The Use of High Strength Steel in Ordinary Reinforced and Prestressed Concrete Beams, by P. W. Abeles. Intern. Assoc. for Bridge and Struct. Eng., Fourth Congress, Prelim. Publication 1952.

(14) Recent Research in Reinforced Concrete and its Application to Design, by A. L. L. Baker. Jour. Inst. Civ. Eng., Feb. 1951.

(15) Mag. of Conc. Research, Vol. IV, No. 12, April 1953, pp. 127–142.

(16) Further Notes on the Principles and Design of Prestressed Concrete, Part 10, by P. W. Abeles, Civ. Eng. and Pub. Works Rev., July, 1951.

(17) The Use of High Strength Steel in Ordinary Reinforced and Prestressed Concrete Beams, Supplement, by P. W. Abeles. Intern. Assoc. for Bridge and Struct. Eng., Fourth Congress, Final Report, 1953.

(18) Fatigue Tests on Partially Prestressed Concrete Members, by P. W. Abeles. Intern. Assoc. for Bridge and Struct. Eng., Fourth Congress, Final Report, 1953.

Concrete Roads.

Two brochures on concrete roads have been issued by the Cement and Concrete Association, one on Earthworks, Subgrades, and Bases and the other on Design of Slabs. Both brochures are based on the experience gained in this country and abroad, and deal with the subject in a simple and practical manner. Modern practice is described, with tables relating to methods of compacting soil and recommended thicknesses of slabs and quantities of reinforcement for roads to carry various densities of traffic. The brochures are available gratis from the Association at 52 Grosvenor Gardens, London, S.W.1. The Road Research Laboratory of the Department of Scientific and Industrial Research at the same time published Road Note No. 19 (H.M. Stationery Office. Price 9*d*.), which contains the same tables and similar information to those given in the Association's brochure on Slab Design. [This is a quick and unexpected example of the duplication of information mentioned in the Editorial note of our April number.]

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Tests on Beams with Pre-tensioned Wire.

Tests have been made by the Civil Engineer's Department, Eastern Region, British Railways, on beams prestressed with pre-tensioned smooth wires of 0·276 in. diameter. The first tests, made in 1952, showed that this wire had satisfactory bond in high-strength concrete, and further tests were made in 1953–54 in collaboration with the Railway Research Department.

In the first tests six rectangular beams were tested. Their lengths varied from 10 ft. 6 in. to 21 ft. 6 in. and they were 8 in. wide by 12½ in. deep in cross section. Two concentrated loads were applied, each 1 ft. 9 in. on either side of the centre of the length of the beam. The spans were 9 ft. 6 in., 13 ft. 6 in., and 19 ft. 6 in. respectively. Six tensioned wires were placed 1¾ in. from the bottom and two tensioned wires 1¼ in. from the top. Some of the beams had six additional non-tensioned twin-twisted wires 2¼ in. from the bottom.

No stirrups were used and in the case of the beams with a high percentage of steel early failure occurred due to shear. The 15-ft. 6-in. beams and the 10-ft. 6-in. beams with tensioned wires only showed a satisfactory bond of the wires. In the longer beam the load causing failure appreciably exceeded the calculated load based on a tensile force in the steel of 101 tons per square inch and a compressive strength in the concrete of 6000 lb. per square inch. In the shorter beams failure due to combined shearing and bending forces took place due to the increased shearing stresses, but no slip of the wire occurred and the load at failure was in accordance with the calculated value. One of the beams spanning 19 ft. 6 in. having a larger amount (1 per cent.) of steel failed due to combined shearing and bending forces at a load which was only 90 per cent. of the calculated maximum load, but no slipping of the wires was observed.

In 1953 a beam was tested in which six wires were tensioned and six single wires were not tensioned. In this case failure due to shearing was avoided by the provision of stirrups, but apparently slipping of the non-tensioned wires occurred and plain straight untensioned wires of 0·276 in. diameter are therefore not recommended. It may, however, be concluded that smooth tensioned wires of 0·276 in. diameter spaced $1\frac{1}{8}$ in. apart will ensure sufficient bond resistance in concrete with a compressive strength of 6000 lb. per square inch when tested on cylinders.

It has been ascertained by many tests that pairs of untensioned twisted wires of 0·2 in. diameter ensure satisfactory bond resistance; this appears to be the case also with wire of 0·276 in. diameter, although this has not yet been proved.

Fatigue tests were made in 1954 on beams 15 ft. 6 in. long. On two beams 1,000,000 repetitions were applied of loads corresponding to a compressive stress of 10 lb. per square inch and a nominal tensile stress of 600 lb. per square inch. Previously the beams were statically loaded until fine cracks had developed, and these cracks were invisible after completion of the fatigue tests. In subsequent static tests the wires fractured under the maximum calculated load.

In another test one million repetitions of load were applied as before on an uncracked beam. Afterwards nearly 9 million repetitions of load were applied to the beam in a cracked state. The lower limit of load corresponded to a compressive stress of 270 lb. per square inch, and the upper limit to a nominal tensile stress gradually increasing from 650 lb. to 1270 lb. per square inch, when the load corresponded to two-thirds of that at which in a static loading test the beam failed due to fracture of the wire. The cracks were invisible at zero stress up to and including the eight millionth repetition. It is thought that these tests prove the suitability of pre-tensioned wire of 0·276 in. diameter, except that single untensioned wires are not satisfactory and provided that the minimum spacing be observed. Also, indented wires would be more satisfactory.

The beams were made by Anglian Building Products, Ltd., at whose works the tests were made, and the wire was supplied by Messrs. Richard Johnson & Nephew, Ltd. The arrangements for the tests were made under the direction of Mr. J. I. Campbell, M.I.C.E., at that time Chief Civil Engineer, British Railways, Eastern Region.

Bridges in Essex.

In connection with the electrification of the Shenfield-Chelmsford-Southend lines the overhead clearances of eleven bridges over the track are to be increased. The spans will vary between 27 ft. 5 in. and 46 ft. 4 in. The decks will be of partially prestressed composite concrete slab construction similar to a number of bridges reconstructed in connection with the electrification of the Manchester-Sheffield-Wath lines. The design has been used during the last six years for some thirty road bridges over railways in Great Britain and is composed of precast prestressed beams and additional concrete placed in situ. The precast I-shape beams, which contain pre-tensioned wire of 0·276 in. diameter, are made in a factory on the long-line system; when placed side by side they support, without propping, the concrete cast between and above them, and the resulting slab acts as if the entire construction were prestressed. The design of the beams has been standardised for road bridges with spans between 20 ft. and 50 ft.

Electrical Curing of Concrete.

METHODS of curing concrete by electrical heating in use in the northern districts of Japan are described by Mr. Chuzo Itakura in the Journal of the American Concrete Institute, of which the following is an abstract.

Experiments show that the modulus of rupture and compressive strength of concrete cured with electricity are higher than with ordinary curing at an early age, but for older concrete they are almost the same. Pull-out tests of a bar embedded in concrete show that when the temperature difference between the bar and the concrete is less than 50 deg. F. bond stress is increased in electrically-cured concrete compared with that for curing in air. When the temperature difference was 77 deg. to 79 deg. F. with the temperature of the reinforcement bar as high as 176 deg. or 194 deg. F., the bond stress was greatly reduced. Care must be taken in the electrical curing of reinforced concrete, and it is dangerous to heat reinforcement bars for a long time. The temperature of the concrete during electrical curing must be kept below 104 deg. F. and usually below 86 deg. F.

The electricity consumed is about 23 to 30 kw.h. per cubic yard of concrete if the whole of the concrete is heated and 0·65 to 0·90 kw.h. per square foot of shutter area for partial heating. The cost of electrical curing is said to be 7 per cent. to 10 per cent. of the construction cost of plain concrete, and 10 per cent. to 15 per cent. of the construction cost of reinforced concrete, including 20 per cent. capital cost for equipment. In general, the cost is affected by the size, shape, and plan of structures, and climatic conditions, and is estimated to be about 30 to 40 per cent. of that for steam curing or heated enclosures, including the cost of construction of any temporary covering that is required.

To get the best effect with the lowest consumption of power it is necessary to use the least possible quantity of mixing water and well-graded fine and coarse aggregates, to use the smallest amounts of admixtures, and to raise the temperature of the concrete in cold weather.

A single-phase or a three-phase transformer can be used. A transformer adjustable from 10 or 20 volts to several hundred volts is desirable. If a transformer with this wide range of voltage is not available, an ordinary transformer may be used by adjusting the terminals on the high-tension side. Two-pole or three-pole switches corresponding to the single-phase or three-phase source, with capacities of 100 to 200 amperes and several hundred volts, are necessary in numbers commensurate with the capacity of the transformer. A circular-type voltmeter for a switchboard or a precision box-type voltmeter is required. The ammeter should be of the circular type for a switchboard or an ammeter for mains supply. The latter is convenient to measure electricity in main wiring and in individual electrodes. An accumulating-type wattmeter is desirable. A special resistance thermometer with terminals embedded in the concrete is best, but common mercury or alcohol thermometers may be used. Flood lighting or electric torches are necessary to permit observation and measurement of the electricity and of the temperature of the concrete at night.

If copper electrodes are used, resistance between the electrodes and the concrete increases and a small direct current may flow between the copper and the iron which may be present in concrete. Sheet or strip iron, iron bars and iron wire are therefore used. For joining the electrodes to branch wires, Nos. 8 or 10 B.W.G. galvanised or plain wires are convenient, especially when the electrodes are embedded in the concrete. Old wire can be used if any loose rust is removed. For the main wiring and connecting the transformer to the electrodes through switches, both soft and hard copper wire are serviceable in single and multiple circuits and should be well insulated. Branch wires connecting the main wiring and the electrodes are preferably insulated, but occasionally uninsulated wires may be used. Many soft thin wires are better than a few large ones, since they expedite the joining of the electrodes, but it must be ensured that they can carry the power required. Safe loads of wires and electrodes must be strictly observed, but if the period of use is short an overload of 20 to 30 per cent. may be allowable.

Electrodes may be (1) Placed on the inner face of the shutters, (2) Placed on

the surface of the concrete, or (3) Embedded in the concrete. Iron wire or sheet-iron electrodes are fixed to the inside surface of the shuttering, ordinarily in a vertical position, with nails, staples, or bolts. By this method electrodes can be accurately spaced and the wiring is not disturbed by concreting. The disadvantage is that marks of the electrodes remain on the surface of the concrete after removal of the shuttering. In partial heating, in which adjacent electrodes are used reciprocally from anode and cathode, there may be considerable leakage of electricity through the shutters when the boards are wet. The electrodes may be laid on the top surface of the concrete and fixed to the side shutters. When the distance between side shutters is large, placing of the electrodes before concreting is difficult; in this case the electrodes are placed after most of the concreting is finished and are embedded in the concrete to a minimum depth of 2 in. The electrodes must be embedded deep enough to prevent their being disturbed and to ensure good contact with the concrete.

Completely embedded electrodes are in the form of rough nets. They can be easily stretched between the shutters at the position of the tie-wires or bolts (*Fig.* 1). If the safe electrical load for the electrodes is exceeded they overheat. If an electrode is too long the electricity flowing in the concrete near the feed end is greater than that near the other end, so a uniform temperature rise in the concrete becomes impossible. The length of the electrodes must therefore be limited.

The general arrangement of electrical equipment for single-phase and three-phase supply is shown schematically in *Fig.* 2. For single-phase electricity each electrode must be connected with a branch wire, coming alternately from the main wire. When a three-phase source is used, each electrode must be connected with a branch wire so that in each series of three electrodes the fuse is connected with one pole, the second and third with the other pole, and so on in sequence, as shown in *Fig.* 2. To join a branch wire to a sheet-iron electrode, bolts are convenient. When there are many electrodes and the branch wires are thin and soft, the branch wire is wound tightly around the electrode.

In many cases nets of electrodes are

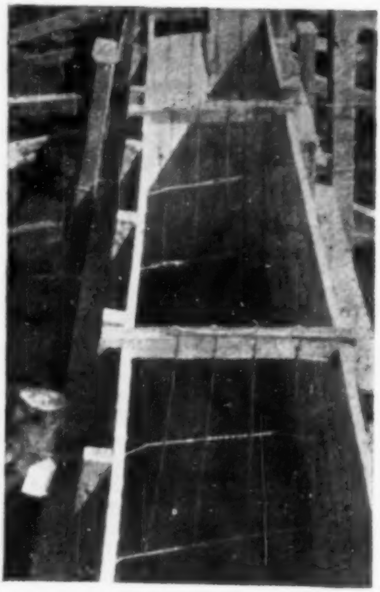

Fig. 1.—Electrodes in the Form of a Net.

placed horizontally or vertically before concreting. In horizontal work electricity passes vertically through the concrete between the upper and lower horizontal electrodes. The lower nets are set before beginning to place concrete and the upper nets are set immediately upon completion of concreting or, if more convenient, during concreting. If the nets of electrodes are stretched too tightly gaps may be left under them as the concrete sets, so the nets should be loosened slightly a few hours after concreting is finished.

When the depth of concrete placed on successive days is not the same, the space between the nets of electrodes may vary with resultant changes in power consumption and in the temperature of the concrete. On the other hand, if power consumption and the rise of the temperature of the concrete are to be constant, the depth of a day's concreting is limited and placing must not be stopped. If there is a difference between the areas of the upper and lower nets the heat is concentrated in the smaller net, so that the ratio of the area covered by small and

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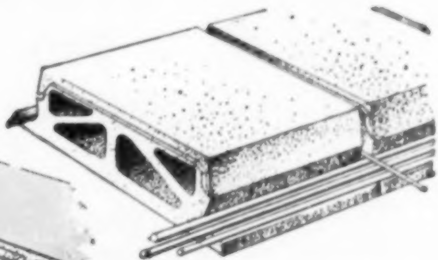

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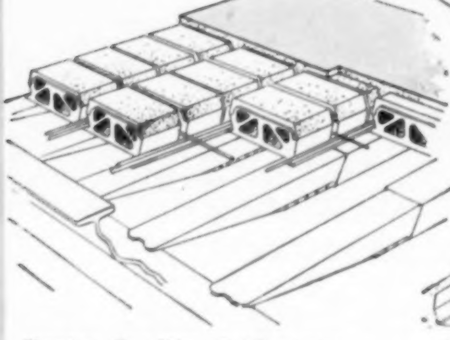

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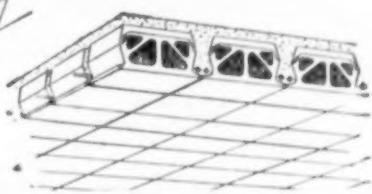

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Some views of the open-air swimming pool at the Skegness Holiday Camp.
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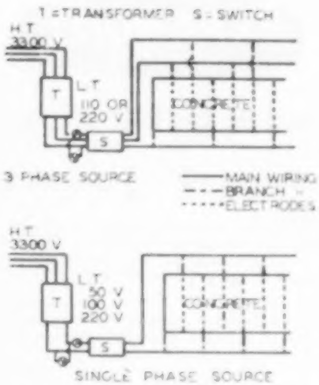

Fig. 2.—Arrangement of Electrical Equipment.

large nets must be kept within about 1 : 1·5.

In vertical work the electricity passes horizontally through the concrete between vertical nets of electrodes embedded in the concrete. The nets can be set before concreting, so the work of embedding the electrodes in the concrete is avoided. When successive lifts of concrete are placed the new concrete can be heated by cutting off the electrodes at the surface of the old concrete. If branch wires are joined to the electrodes at their upper ends it is necessary to draw out the electrodes by perforating the shutter board.

The whole volume of concrete may be heated so that the temperature is raised uniformly during setting and hardening. This method is applicable to plain concrete, and to reinforced concrete if the spacing between reinforcement is large or if the cross section is thick and has an initial temperature below 50 deg. F. Alternatively when necessary part only of the concrete, usually the exposed surface, is treated. When the cross section is less than 10 in. to 12 in. thick the results obtained are almost the same as those obtained by treating the whole of the concrete. In a large member the sudden rise in the temperature of the concrete near the electrodes is pronounced, and when the electricity is switched off a sudden drop of temperature in that part occurs because of the small enthalpy of the electrodes. In this method many linear sources of heat are installed at the cooling surface, and in the inner portion the generation of heat during setting is gradual and delayed. If the initial temperature of the concrete is more than 58·5 deg. F. this method can be used safely. There is a distinct possibility that the concrete near the electrodes may be hardened before the concrete between the electrodes, and there may occur some initial strains unfavourable to the fresh concrete; the switches should therefore be operated at intervals of thirty to forty minutes. This method is commonly employed for concrete with a great deal of reinforcement.

In general, with metal conductors the current becomes weaker with the rise in temperature, but when ion conduction is involved, as in the case of cement paste, the electrical resistance decreases and the current increases with the rise in temperature. This correlation between electrical current and temperature continues until setting of the cement paste begins and the temperature reaches a maximum. As setting begins, electrical resistance changes to that of a solid and the volume of current decreases. The temperature drops with reduction of the current, causing delay in the setting. If the voltage is constant, time-current and time-temperature curves are uniform, but if the current and the temperature change suddenly the current must be switched off and the work inspected carefully. In a large mass of concrete the generation of heat during the setting of the inner concrete becomes important, and the temperature rises or remains constant for a long time after the current is switched off.

Concrete mixed with salt water has small electrical resistance, so that the quantity of current flowing in the concrete is great and the rise of temperature is slow; and special care is also necessary in concrete work in coal or metal mines and along the sea coast.

The operations should be arranged to secure uniform rise of temperature in every part of the concrete. It is necessary (1) To space similar electrodes closely (generally not more than 6 in. to 8 in. apart) and to space farther apart electrodes of dissimilar nature to allow the temperature to rise more slowly; (2) To turn the electricity on and off frequently; (3) To place the concrete so as to raise the temperature uniformly and

to reduce the necessity of raising the temperature by electricity; and (4) To use the most suitable lengths of electrodes and main leads. When electrodes are laid vertically or horizontally and exceed a certain length (even within the safe value of the electrode), and if the feed ends are all at one side, then the voltage falls at the other end and the current becomes weaker with a delay in the rise of temperature. In such cases the electricity should be fed to the opposite end of every electrode alternately, or the electrodes must be shortened and fed directly. The same considerations apply to the main wires. If the main lead wire is long and has many branch wires attached to it, there is a great reduction in voltage and the rise of the temperature of the concrete is delayed at the end of the main wire. In such cases, the main wiring must be arranged in a loop circuit or the electricity must be supplied by several parallel circuits from the transformer to the switches.

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word : minimum, 7s. 6d. Situations Vacant, 4d. a word : minimum, 10s. Other miscellaneous advertisements, 4d. a word : 10s. minimum. Displayed advertisements, 30s. per column inch. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATION VACANT. Concrete formwork designer-draughtsman required by civil engineering contractors in Westminster. Apply in writing only, stating age, experience, and salary required, to Peter Lind & Co., Ltd., Romney House, Tufton Street, London, S.W.1.

SITUATIONS VACANT. Experienced reinforced concrete designers and detailers required by consulting engineers in their Sunbury office. Five-days' week. Permanent position, good salary and prospects. Apply, stating age and experience, to J. H. Coombs & Partners, Thames Corner, Sunbury-on-Thames.

SITUATION VACANT. Detailer-draughtsman required for varied reinforced concrete work. Permanent, progressive, and superannuated post. 36½ hours' week. Alternate Saturdays. Telephone for appointment (Mr. Disney), the Rom River Co., Ltd., Euston 7814.

SITUATION VACANT. Reinforced concrete detailer required for consulting engineer's office. Good experience in reinforced concrete essential. Five-days' week. Apply in writing, giving details of experience and salary required, to F. J. Samuely, 8 Hamilton Place, London, W.1.

SITUATIONS VACANT. The Trussed Concrete Steel Co., Ltd., have vacancies in their London, Birmingham, Glasgow, and Manchester offices for reinforced concrete designers and detailers. Five-days' week. Pension scheme. Apply, giving full particulars of age, education, and previous experience, to the Secretary, Truscon House, 35–41 Lower Marsh, London, S.E.1.

SITUATIONS VACANT. Reinforced concrete designers and draughtsmen with at least two years' office experience required. Interesting work and good prospects. Five-days' week. Apply in writing, stating full particulars, to Christiani & Nielsen, Ltd., Romney House, Tufton Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers and detailers required by consulting engineers for varied and interesting building frames and industrial structures. Salary £500 to £700 per annum, according to experience. Apply with details to John F. Farquharson & Partners, Chartered Structural Engineers, 34 Queen Anne Street, London, W.1. LANgham 6081.

SITUATIONS VACANT. Reinforced concrete designers and detailers required by consulting engineers to work in Surrey office. Good working conditions and five-days' week. Salaries £500–£900 per annum, according to experience and ability. Permanent positions with excellent prospects. Apply, giving full details, to Box 4138, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Tarmac, Ltd., require senior estimator for concrete products. Able to read drawings and prepare own quantities. Up-to-date experience essential. This appointment is permanent and well paid. Write, giving full details of experience, and salary required, to Clerical Manager, Tarmac, Ltd., Ettingshall, Wolverhampton.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., have vacancies for reinforced concrete designers and detailers, with experience, in their Stafford, London, Liverpool, Bristol, Newcastle-upon-Tyne, and Glasgow offices. Staff pension scheme, and five-days' week. Apply in writing to Chief Engineer, Stafford.

SITUATION VACANT. Estimator required for reinforced concrete light-framed structures, including hollow tile floors, etc. Apply in writing, giving brief details of experience and salary required, to The Helical Bar & Engineering Co., Ltd., 82 Victoria Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designer-draughtsmen required by Ashmore, Benson, Pease & Co., Stockton-on-Tees. Applicants should be fully experienced in either designing or detailing reinforced concrete structures, foundations and other civil work. Company's flat, pension scheme, technical library, and sports facilities available. Apply, stating age, experience, etc., quoting reference "D", to Staff Personnel Officer.

SITUATIONS VACANT. Reinforced concrete designer-detailers wanted by British company in Kampala (Uganda) and Dar-es-Salaam (Tanganyika). Candidates should have had at least five years' experience. Free air passages, two months' paid home leave by air each 2½ years, bonus and pension schemes. Applications will be treated in strict confidence. Write Box AP/125, 95 Bishopsgate, London, E.C.2.

SITUATIONS VACANT. A large firm of reinforced concrete engineers in Rhodesia and the Union of South Africa has vacancies for reinforced concrete designers and detailers with experience. Apply in writing, with full details of qualifications and experience, age, and salary required, to Box R4/4/55, 95 Bishopsgate, London, E.C.2.

SITUATION VACANT. Civil engineering assistant required for design work by London consultants. Previous experience in water engineering advantageous. Graduate with design experience suitable. Remuneration not now stated as it will be based on qualifications and experience. Please indicate technical education, age, and previous experience, to Box 4141, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

CITY OF CARDIFF

APPOINTMENT OF STRUCTURAL ENGINEERING ASSISTANT

Applications are invited for the following appointments in the City Surveyor's Department :—

Structural Engineering Assistant, A.P.T. New Grade 6 (£825–£1,000 per annum).

Candidates should possess the minimum qualifications and experience prescribed by the National Joint Council for Local Authorities' Administrative, Professional, Technical and Clerical Services for posts in the above-mentioned Grade.

General Conditions of Appointment may be obtained from the undersigned.

The Council will assist in providing housing accommodation for the successful applicant.

Applications, accompanied by the names and addresses of three referees and endorsed "Structural Engineering Assistant, A.P.T. New Grade 6", must be delivered to me not later than the 26th May, 1955.

S. Tapper-Jones,
Town Clerk.

City Hall,
Cardiff.

April, 1955.

INTERMEDIATE DRAUGHTSMEN

Required for expanding office of Consulting Engineers engaged on varied schemes in Reinforced Concrete and Structural Steel construction. Offering permanent posts with :

(i) Salary commensurate with experience.
(ii) Favourable working hours, with optional overtime.
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(iv) Prospects of rapid advancement.

Applications immediately to Box 4156, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

WEST SUSSEX COUNTY COUNCIL

County Architect's Department

Applications are invited for the following appointment :—

ASSISTANT STRUCTURAL ENGINEER at a salary in accordance with A.P.T. Grade VI (£825 to £1000 per annum).

Further particulars should be obtained from the County Architect, County Hall, Chichester, to whom detailed applications must be submitted not later than the 2nd June, 1955.

T. C. HAYWARD,
Clerk of the County Council.

County Hall,
Chichester.
7th April, 1955.

SITUATIONS VACANT. Design assistants required by Westminster professional firm. Range of work is wide and posts available cover both medium and senior status. Salary will be considered on qualifications and experience basis. Other facilities include staff pensions, alternate Saturdays, and canteen. Please give usual details through Box 4143, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Chief designer-draughtsman (capable of controlling medium-sized drawing office) required for London head office of a firm of reinforced concrete engineers and contractors. Applicants should be qualified and possess a wide experience of building frame design and knowledge of precast and prestressed concrete construction. The post is permanent and pensionable, two weeks' holiday this year, and carries a salary of £950 to £1,100 per annum, according to qualifications and experience. Write full particulars to Box 4144, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Applied building research. Substantial building organisation requires a chief research assistant with forward outlook, for development of new materials and techniques and supervision of concrete and prestressed concrete quality controls. Suitable qualifications and experience required. Five-days' week and pension scheme. Headquarters near London. Apply, stating salary expected, to Box 4145, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. THE STANTON IRONWORKS COMPANY, LTD., has a vacancy for a young man in its Concrete Research Department. Applicants should be up to the standard of the Ordinary National Certificate in engineering or equivalent. Some experience of concrete would be an advantage. There are excellent prospects and the post is superannuated. Write to the STAFFING OFFICER, STANTON IRONWORKS COMPANY, LTD., P.O. Box No. 3, near Nottingham.

SITUATIONS VACANT. Z. PICK, CONSULTING ENGINEER, requires in his London office, junior and senior designers and detailers for reinforced concrete work. Five-days' week. Permanent positions. Apply to 42 Ferncroft Avenue, London, N.W.3.

SITUATION VACANT. Technical officer required for Birmingham area. Duties will include concrete quality control and works' investigation. Applicants should be 20–25 years of age and willing to continue studies in civil engineering or industrial chemistry. Good prospects of advancement. Apply in own handwriting, stating education, qualifications, and experience, to Box 4146, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers and site travelling engineers with good theoretical knowledge and experience in industrial design required by civil engineers and contractors in Westminster. Good prospects in expanding business. Pension scheme. Staff canteen. Apply in writing, stating age, qualifications, experience, salary required, to TILEMAN & CO., LTD., Romney House, Tufton Street, London, S.W.1.

SURREY COUNTY COUNCIL

Applications invited for appointment of ASSISTANT STRUCTURAL ENGINEERS GRADE IV, £675 - £30 to £825 p.a. plus London allowance. Must be experienced in design and detailing of reinforced concrete building frames and/or structural steelwork. Preference given to those who have passed final exam. of Institution of Civil or Structural Engineers.

Full details and present salary, with three copies testimonials, to COUNTY ARCHITECT, County Hall, Kingston, by 21st May, 1955.

SITUATIONS VACANT. Reinforced concrete draughtsmen with experience in detailing industrial structures required by civil engineers and contractors in Westminster. Good prospects in expanding business. Pension scheme. Staff canteen. Apply in writing, stating age, experience, salary required, to TILEMAN & CO., LTD., Romney House, Tufton Street, London, S.W.1.

SITUATIONS VACANT. Draughtsmen and design assistants required by consultants in Westminster. Several vacancies now available due to extension of work and staff transfers to more responsible positions. If you would indicate your technical education and experience, appointments could be arranged on basis that remuneration, though not now quoted, would be on basis of ability and experience. Positions are permanent and eligible for staff pensions. Box 4147, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Civil and structural engineering assistants, experienced in reinforced concrete and structural steel work, required. Knowledge of design will be an advantage but not essential, depending on positions. The work is varied and offers scope for initiative. Five-days' week. Pension scheme. Good salaries will be offered to suitable applicants, who should state experience and salary required, to ENGINEERING DEPARTMENT, FARMER & DARK, Romney House, Tufton Street, London, S.W.1.

SITUATION VACANT. Structural Engineer. Due to large increase in work of structural development department, required, for company in Dagenham area, assistant to chief engineer. Must be fully qualified structural engineer with practical experience of design. Age 25 to 35. Adequate commencing salary, with ample scope for initiative and exceptional prospects for the right man. Write in confidence, stating qualifications, experience, and salary required. Box 4151, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Experienced designer required by the INDENTED BAR & CONCRETE ENGINEERING CO., LTD., capable of designing and detailing all classes of reinforced concrete structures. Some supervision would be given. Apply, giving details of experience and salary required, to 171 Victoria Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete designer-draughtsman. Apply, giving experience, age and salary required, to HOLST & CO., LTD., 10 Waddington Street, Durham.

SITUATIONS VACANT. Structural or civil engineering draughtsman required for consulting engineers' office in Newcastle-upon-Tyne. The work is of an interesting and widely-varied nature. Write, giving details of age and experience, to R. T. JAMES & PARTNERS, Clavering Place, Newcastle-upon-Tyne, 1.

SITUATION VACANT. Assistant engineer or draughtsman familiar with foundation work required by foundation engineers for drawing office. Permanent position. Salary according to age and experience. Apply in own handwriting to Box 4142, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

W. & C. FRENCH, LTD., have a vacancy in their drawing office at Buckhurst Hill for an assistant experienced in detailing reinforced concrete work. Some knowledge of design desirable. Reply, stating age, experience, and salary required, to Personnel Manager, Buckhurst Hill, Essex.

(Continued on page lxx.)

MISCELLANEOUS ADVERTISEMENTS

(Continued from page lxix.)

SITUATION VACANT. Senior estimator required by leading asphalt company for London office. Must be fully experienced in trade and conversant with sources of supply. Permanent and pensionable position. Salary, commensurate with experience up to £1,200 per annum. Own staff advised. Apply to Box 8565, CHARLES BARKER & SONS, LTD., 31 Budge Row, London, E.C.4.

SITUATIONS VACANT. PIERHEAD, LTD., prestressed concrete engineers, require designer-draughtsmen for work on both prestressed and normal reinforced concrete. Opportunity to gain useful experience in this field of construction. Pension scheme, sports club. Apply, giving fullest particulars and salary required, to CHIEF ENGINEER, PIERHEAD, LTD., Faggs Lane, Feltham, Middx.

SITUATIONS VACANT. Dock and harbour design assistant required, Westminster area. Position would be as a senior assistant, but there is shortly a vacancy also for a less experienced man. Please give indication of education, any professional qualifications, and experience to Box 4148, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Chief draughtsman required by firm engaged in design of reinforced concrete, and supplying and fixing steel reinforcement, to take charge of rapidly expanding drawing office. Applicants must have had similar experience and preferably have some commercial knowledge. Very good prospects for right man. Five-days' week, canteen, pension scheme, etc. Write, stating age, experience, and salary required, to Box 4149, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete assistant, with some knowledge of design, required for engineer's office of a firm of architects and engineers. Interesting prospects to man requiring varied experience. Write, giving full details and salary required, to ROLAND WARD & PARTNERS, 29 Chesham Place, London, S.W.1.

SITUATION VACANT. Precast concrete general works manager required to take complete works control, responsible only to director. Good, sound, practical man with general all-round experience including prestressed techniques and cast stone, together with mass-production methods. Age 35–50. State full details of experience in chronological order. No one need apply who is not capable of earning or has earned £1,500 per annum or more. Only fully detailed applications will be considered. Write Box 867, WALTER JUDD, LTD., 47 Gresham Street, London, E.C.2.

SITUATIONS VACANT. Consulting engineer, Westminster, requires reinforced concrete designer-draughtsmen. Also required, designers with experience in shell roofs. Apply, giving full particulars, to JAMES E. WARDROPPER, 116 Victoria Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designer and detailer-draughtsmen required by leading firm of constructional engineers for their London office. Applicants should preferably be experienced in frame and floor construction. Good salaries for the right applicants. Working conditions are good and there is a non-contributory pension scheme. Present staff notified of these vacancies. Write, giving full details of age, qualifications, experience, and salary required, to Box 4154, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers and detailers required in Keynsham (Bristol) office of reinforcement specialists. Opportunity for varied experience to suitably qualified applicants. Draughtsmen without specialist experience considered for training as detailers. Box 4155, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designer-detailers wanted by British Co. in Mombasa (Kenya), Kampala (Uganda), and Dar-es-Salaam (Tanganyika). Candidates should have had at least five years' experience. Salary according to ability and experience. Free air passages, two months' paid home leave by air each 2½ years, bonus and pension schemes. Applications will be treated in strict confidence. Write Box R13/4/55, 95 Bishopsgate, London, E.C.2.

COUNTY OF LEICESTER

Applications are invited for the following posts in connection with the proposed Cavendish Bridge Diversion (Route A.6).

(a) **Resident Engineer**

Applicants must be qualified Civil Engineers and have had considerable experience in the supervision of bridge construction, particularly in prestressed concrete, earthworks, and road construction by modern methods.

The inclusive salary offered is £1,400 per annum.

(b) **Clerk of Works**

Applicants must have had considerable experience in road and bridge works, including setting out of works and control of high-grade concrete for reinforced concrete structures.

Experience in the control of concrete for prestressed concrete structures would be an advantage.

The inclusive salary offered is £800 per annum.

These appointments are for approximately twenty four months subject to two months' notice on either side.

Forms of application can be obtained from R. W. GRIGSON, T.D., B.Sc., M.I.C.E., the County Engineer and Surveyor, County Offices, Grey Friars, Leicester, to whom they must be returned not later than Saturday, 4th June, 1955.

SITUATIONS VACANT. Several civil engineering draughtsmen required for work in London office of large oil company. Experienced in the preparation of designs and calculations for foundations and steel or reinforced concrete structures, also the preparation of specifications and bills of quantities. Candidates with at least O.N.C. or equivalent and a good basic training in civil engineering. They should also be willing to serve overseas if required. Salary £700–£900 per annum according to qualifications and experience. Pension fund. Write in detail, quoting No. 467, to Box 8619, CHARLES BARKER & SONS, LTD., 31 Budge Row, London, E.C.4.

SITUATIONS VACANT. Reinforced concrete detailer-draughtsmen required by civil engineering contractors in Westminster. Two Saturdays free each calendar month, canteen facilities, and pension scheme. Apply in writing only, stating age, experience, and salary required, to PETER LIND & CO., LTD., Romney House, Tufton Street, London, S.W.1.

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